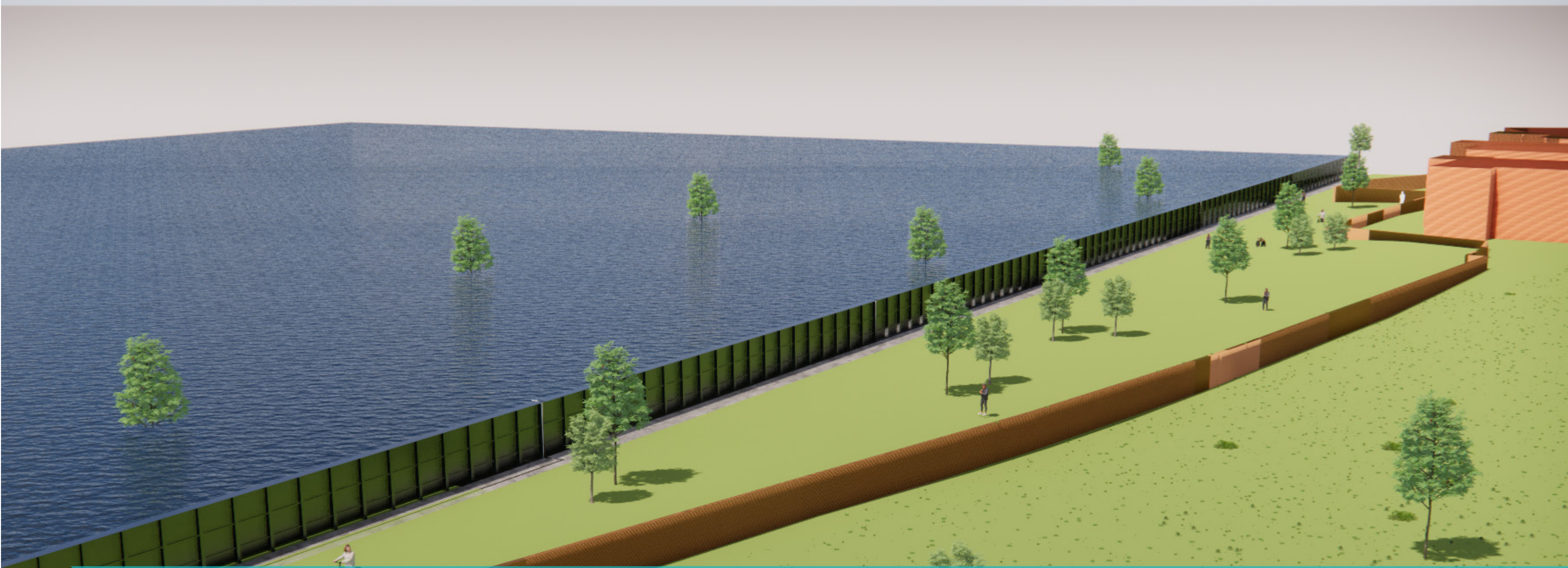
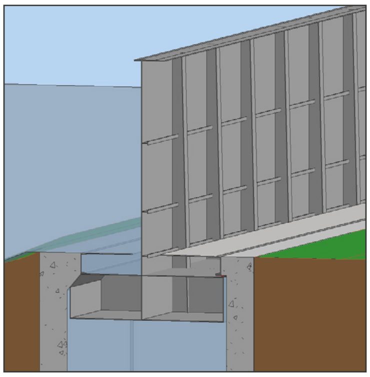


Design Study on the Feasibility of a Self-closing Flood Barrier | 2024

S. S. Jhinkoe-Rai

A Case Study of the City of Arcen, Limburg



MSc Thesis
Delft University of Technology
In collaboration with Iv-Infra B.V.

February 2024

TU Delft Delft University of Technology

Ingenieurs met passie voor techniek

Design Study on the Feasibility of a Self-Closing Flood Barrier

A Case Study of the City of Arcen, Limburg

by

S.S. Jhinkoe-Rai

to obtain the degree of Master of Science
at the Delft University of Technology,
to be defended publicly on Wednesday 28 February, 2024 at 13:30 PM.

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Preface

This thesis is part of a graduation project for obtaining the degree of Master of Science (MSc). It is conducted as part of the master's degree in Structural Engineering at the Delft University of Technology with a specialisation in hydraulic structures.

This graduation project is performed in collaboration with the engineering firm Iv-Groep b.v. Iv-Groep is a consultancy and engineering firm that operates worldwide. They provide their services in the following key markets: Installation Technology, Handling, Infrastructure, Offshore and Energy, Maritime and Water. Iv-Infra b.v. is part of Iv-Groep b.v. and operates in the Infrastructure market. Within this division of Iv-Groep b.v. I will be graduating.

Using this opportunity, I would like to thank my supervisors Mark Voorendt and Sander van Nederveen and chair graduation committee Matthijs Kok from TU Delft and my supervisor Ruud Nooij from Iv-Infra b.v. for their supervision and advice during my graduation project.

S.S. Jhinkoe-Rai
Delft, February 2024

Abstract

Flood risks and its consequences become more and more challenging and are demanding for the currently present dike systems and hydraulic structures in the Netherlands. Major adjustments in the Flood Protection Program lead to stricter requirements for flood protection systems which are able to still protect the hinterland in case of an expected high water level. This automatically raises the question of the possibility of innovatively adjustable or temporary flood protection systems. The recent river floods of 2021 in Belgium, Germany and The Netherlands justify the need for adjustments. Because of the major dike reinforcement programme, it is more likely that a standard earthen dike will not be possible everywhere, due to site-specific characteristics such as available space or protected townscape. At these specific locations, a self-closing flood barrier can offer a solution, but is relatively new within the range of flood defence systems and can therefore still be customised for specific areas, where it does not yet have a reference for. For example, current self-closing flood barriers are so far applied with a relatively small retaining height. Furthermore, the design of these barriers are not applicable in areas where support structures above ground surface level are prohibited. The goal of this design study is to gain insight in the feasibility of applying a self-closing flood barrier also in areas where spatial quality plays an important role and where the hydraulic boundary conditions demand for heavier structures. With the increase of densely built areas nowadays, this concept is promising and an interesting alternative to consider.

In this thesis the objective is to develop a customised design of an adaptive self-closing flood barrier in the Netherlands. The design of the self-closing flood barrier starts with selecting a suitable case study which might present a location in need for such a hydraulic structure, which in this case is the city of Arcen in Limburg along the Meuse. Analysis of the location presents the concerns and the focus points to which should be prioritised. The city of Arcen is characterised by its cultural-historical values and its strong connection to the Meuse. For this reason the city attracts many tourists throughout the year. This makes it difficult to integrate a conventional water barrier such as an earthen dike for example. From the system analysis the site-specific, functional and structural requirements follow. With this basis of the design, the process continues with inventing solutions that comply to this set of requirements and provide the highest value for solving the problem.

Societal and economic aspects have not been taken into account in the design process. For the design purpose software with statistical data such as Hydra-NL was used to provide site characteristics, hydraulic and geotechnical boundary conditions and literature for information on currently applied barrier types and drive mechanisms. Furthermore, the design process has been completed with the help of the Design Guide Hydraulic Structures of Rijkswaterstaat and the Eurocode, from which a design originated in which stability and strength requirements have been met. The design resulted in a floating barrier of steel with a flat gate type. A floating barrier leads to a more simplistic design with less mechanical parts. Furthermore the selection of a flat gate type is in line with integrating the structure in the area without affecting the area as much as possible during the use of the structure. In the design the focus lies on the functionality and the structural integrity with an in-depth look at adaptability and integrability.

The result shows that the design of a floating flat barrier is feasible as a self-closing structure in the city of Arcen. However, the location of the structure does require some remarkable features such as the absence of townscape obstructing elements which means a support structure that is fully submerged in the ground. This leads to a heavy support structure because the design water level for retaining water is locally almost 2.5 m above the ground surface. Also, the hydraulic conditions require a large floater, even though the structure is slender. In order to develop a complete design, it is recommended to make a detailed design on the concrete foundation, to do a cost analysis on the construction and materials and investigate how a certain structure is received by the residents of Arcen. This will gain

insight in areas to optimise the design.

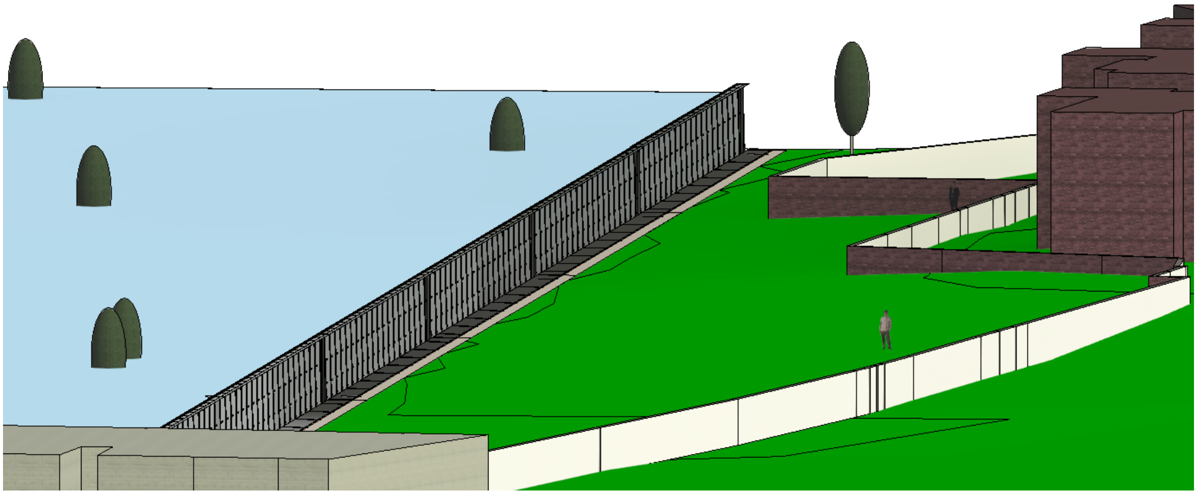

Figure 1: 3D impression of the self-closing flood barrier in the project area

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1 Introduction

1.1. Motivation and relevancy

Since 1 Januari 2017, safety standards for primary flood defenses have been recalibrated in the Dutch Water Act. The Ministry of Infrastructure and Water Management expects that it may take until 2050 before primary flood defenses in the Netherlands meet the new standards. In some areas the requirements for primary flood defenses are being made more stringent due to the risk approach. In addition, the new assessment instruments will include new technical insights that will lead to more strict requirements (Kenniscentrum Infomil Rijkswaterstaat, 2023).

The reason for this recalibration is because of (Kenniscentrum Rijkswaterstaat, 2023):

- the increasing population
- the increase in the economic value of the hinterland
- the desire to fit dike improvements harmoniously into the landscape, where more consideration is given to natural and cultural-historical values
- the need for dike improvements that takes stronger wind and higher water levels into account due to climate change

To elaborate on this, it is expected that in 2035 the Netherlands will have an increase of one million inhabitants (CBS, 2019). 75% of this increase will end up in the large and medium-sized cities (CBS, 2019). This increasing urbanization leads to larger economic value of the areas behind dikes.

A major adjustment in the Flood Protection Program (Dutch: Hoogwaterbeschermingsprogramma) is that the failure probability is considered rather than exceedance probability as was the case in the old program (Kenniscentrum Rijkswaterstaat, 2021). This leads to stricter requirements for flood protection systems which are able to still protect the hinterland in case of an expected flood. This automatically raises the question of the possibility of innovatively adjustable or temporary flood protection systems.

The recent river floods of 2021 in Belgium, Germany and The Netherlands show that these adjustments are indeed necessary. This flood event included a series of severe floods that affected the Dutch province of Limburg in January and February 2021. Heavy rainfall caused the rivers Meuse and Roer to overflow, causing widespread damage to homes, businesses, and infrastructure, see Figure F.1.1. Several towns and villages were evacuated, and the Dutch army was deployed to assist with flood protection and evacuation efforts. The floods caused an estimated € 350 to 600 million in damage, primarily in the 'Geuldal' and several fatalities were reported. The damage was thereby greater than the floods along the Meuse River in 1993 and 1995. Such an event would occur only once every 100 to 1,000 years, particularly in the summer (TU Delft, 2021).

Figure 1.1: Aerial view of Limburg flood in 2021 (METRO, 2021)

Because of new flood protection standards, the Limburg Water Board must ensure that the 185 km of Limburg dikes must be in compliance by 2050 (Waterschap Limburg, 2022). In Figure F.1.2 the tasks for reinforcing the dike segments are indicatively shown.

As can be noted, the planning and realization phase for a dike segment of 7.5 km is planned for 2026 in Venlo. Because this is a major dike reinforcement programme, it is more likely that a standard earthen dike will not be possible everywhere, due to site-specific characteristics such as available space or protected townscape. This could lead to a greater demand for flexible flood barriers and moreover a greater demand for larger contiguous spans of such flexible flood barriers.

Traject	Lengte	Periode *	Opmerking
1 Belfeld	1,0 km	Loopt	Dit traject wordt nu op sterkte gebracht. Meer info.
2 Arcen	5,1 km	Loopt	Bekijk de informatie over dit project.
3 Steyl - Maashoek	0,2 km	Loopt	Bekijk de informatie over dit project.
4 Blerick - Groot-Boller	1,2 km	Loopt	Bekijk de informatie over dit project.
5 Baarlo - Hout-Blerick	4,8 km	Loopt	Bekijk de informatie over dit project.
6 Venlo 't Bat	0,4 km	Loopt	
7 Venlo Genooy	1,9 km	Loopt	Dit traject is onderdeel van de verkenning van Vierwaarden.
8 Venlo - Velden	4,8 km	Loopt	Dit traject is onderdeel van de verkenning van Vierwaarden.
9 Blerick	3,4 km	Vanaf 2026	
10 Venlo	7,5 km	Vanaf 2026	
1 Belfeld	1,0 km	Tussen 2035 - 2050	In deze periode volgt verdere ophoging.

Figure 1.2: Dike reinforcement task for municipality Venlo, Limburg (Waterschap Limburg, 2022)

To elaborate more on the aforementioned, dike reinforcements in the previous section involve measures to primary flood defenses along the Meuse. Some of these areas may have insufficient space to construct new earthen dikes, because these dikes consume a lot of space. In addition to this measure, could dike heightening be a solution but reduces also the water storage capacity of the Meuse, because part of the river's winter bed is being lost, for example. Furthermore, measures on such defences or along these waterways should be done carefully, because they can affect the spatial quality easily. For example, the townscape or livability of the houses might get affected because, for example, gardens are being intersected. On top of that, there is not always support for dike elevations and intersecting measures, because residents like to stay connected with rivers and municipalities would like to preserve natural and cultural-historical values. This is not consistent with the aforementioned measures. Also, demountable flood defences are not always a desirable solution direction, because these type of flexible flood barriers require bearings on surface every few meters, which can detract the townscape as well.

At these specific locations, a self-closing flood barrier can offer a solution. Self-closing flood barriers are relatively new within the range of flood defence systems and in combination with the increase of densely built areas nowadays, this concept is promising and an interesting alternative to consider.

To illustrate, in the Northern Meuse Valley (Dutch: Noordelijke Maasvallei) a dike reinforcement program is ongoing which is in line with the adjustments in the Flood Protection Program. For some specific locations, a self-closing flood barrier was included as an alternative for dike reinforcement. Remarkable is that the residents of the area gave preference to a self-closing flood barrier in comparison with a dike heightening or a permanent transparent structure (Waterschap Limburg, 2020). Almost 50% of the residents preferred a self-closing flood defence. Yet still was not chosen for this alternative by the municipality, because the self-closing flood barrier was too expensive and there were no funding opportunities from the government and the self-closing alternative was characterized as an excessive solution for the particular case(Waterschap Limburg, 2020).

By conducting the design process with a focus on the shortcomings of the self-closing flood barrier, a design can be developed that may have a higher social value and thus gain more support for implementation in the Netherlands.

More insight can be gained into the technical performance of a self-closing barrier, by making an optimal design in line with the flood protection standards and by testing the effect of design variables on the failure requirements from the Dutch Water Act for a self-closing barrier.

So the concept of a self-closing flood barrier is very interesting and promising, but is not sufficiently developed to be widely used. This graduation project further explores the self-closing flood barrier concept to make it a feasible solution and presents a customized design with optimal characteristics of existing designs in order for it to be integrated more in the Dutch water systems as a solution to the increasing challenges concerning flood protection.

In Figure F.1.3 an example of the concept of a self-closing flood barrier is shown.

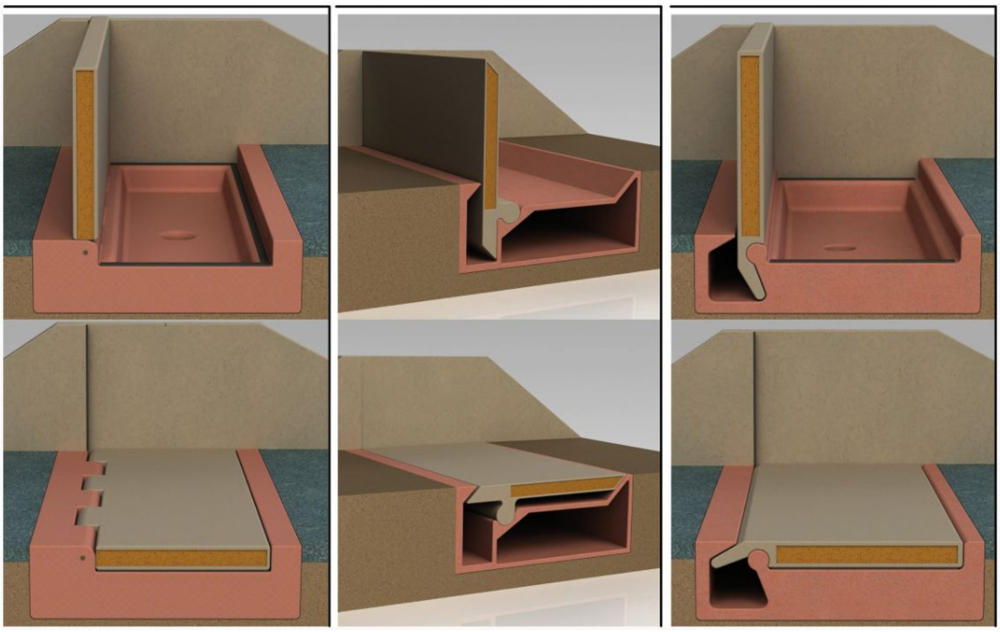


Figure 1.3: Concept of one type of self-closing flood barrier (BFT International, 2020)

1.2. Problem Analysis

1.2.1. General information on self-closing flood barrier

The self-closing flood barriers applied in practice are designed to autonomously be temporarily engaged when hydraulic conditions are extreme threatening hinterland flooding. The need for a temporarily (higher) flood protection height would increase in such a case and this is when the self-closing flood barrier comes into effect. These flood barrier in practice are mostly driven by a buoyant force and conveniently uses the approaching flood wave or rising water level to engage itself (Hyflo BV, 2021).

This type of flood barrier is only a necessity in a specific environment, for example where there is limited space, in residential areas and areas where more consideration is given to natural and cultural-historical values. Also this type of structure could be essential to integrate in quay walls along channels that may or may not be a case at risk for flooding as an additional protection measure.

A remarkable feature of this system is that the structure is not visible when it is not in its retaining function. This means that when it is not active as a flood protection, the structure is submerged into the ground or integrated in an existing structure. For this reason, it does not obstruct traffic routes or the townscape, but it rather fits harmoniously into the landscape taking into account nature and cultural-historical values.

Furthermore, because of the real-time response to potential flooding, the flood risk is more mitigated by increasing the protection height in a relatively short time. This solution may be consistent with new standards for primary flood defences in the Dutch Water Act, since vigilance for expected higher water levels increases, due to consequences of climate change amongst others, like heavy rainfall, storms and rapid melting snow. Important note with such a barrier is that the necessity for human intervention is eliminated, which can be either beneficial or, on the contrary, maybe even unfavourable in some cases.

In addition, this system may be incorporated not only along waterways, but also on a smaller scale to protect directly in front of residential areas in large cities. This is conveniently in line with the increasing urbanization as described in Chapter 1 and the need for additional measures to protect areas with high social and economic values.

Figure 1.4 shows an example of a self-closing flood barrier. More examples of self-closing flood barrier applied in practice are presented in Appendix B.

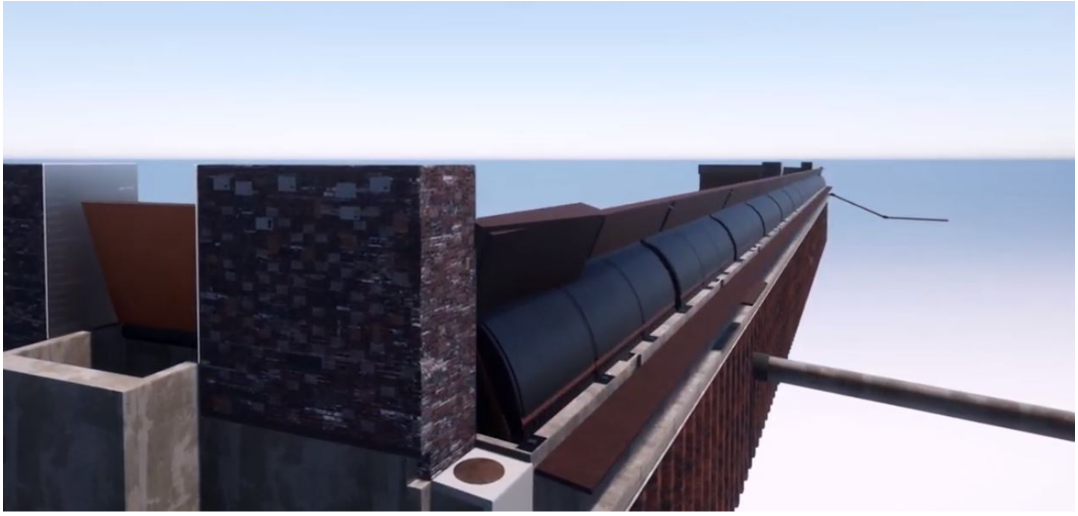


Figure 1.4: Impression of a self-closing flood barrier (Vlotterkering BV, 2021)

1.2.2. Inventory of problems

Beneficence between temporary flood barriers

The manufacturer of one of the existing self-closing flood barriers presents a very own table showing for which flood risk criteria the flood protection system is (equally) favorable compared to other temporary flood barriers. In Figure F.2.3 this table is illustrated in which for several flood risk criteria the flood barriers are scored with respect to each other. The scoring is done by means of grading '0' for no risk to a small risk, 'x' for a small to a moderate risk, 'xx' for a moderate risk to high risk and 'xxx' for a high risk to a very high risk.

It can be observed that the self-closing flood barrier is not necessarily beneficial in comparison to the other temporary flood barriers for the flood risk criteria leakage, sensitivity to maintenance problems and for example deployment. This means that the design of a self-closing flood barrier could be optimized for these criteria, in order for it to be a more desirable solution with respect to other temporary flood barriers. In the next subsections additional problems will be discussed for which the current design might be optimized.

Gradual flood risk criteria	Mobile flood barrier	Demountable barrier	Self closing flood barrier
High water alarm failure	xx	x	0
Failure to respond in time	xxx	xx	0
Operational manpower shortage	xx	x	0
Design weakness	xx	0	0
Possibilty of leakage	xx	x	x
Traffic problems	xxx	xx	0
Closing time	xxx	xx	0
Failure of operational parts	x	x	0
Sensitivity to vandalism	x	x	0
Sensitivity to maintencance problems	x	x	x
Manual operability	xxx	xx	x
Storm operation	xxx	xx	0
Energy consumption	xx	x	0
Recovery rate	x	x	x
Operational control	xxx	xx	0
Deployment	x	x	x
Impediment	xxx	xx	x

Figure 1.5: An analysis of the benefits of three types of temporary flood barriers with respect to each other, regarding several flood risk criteria (Hyflo BV, 2021)

Height of flood barrier and support structures

The self-closing flood barriers in the Netherlands do not exceed the height of 0.8 m. However, changing boundary conditions will in practice require designs of self-closing flood defences with higher retaining heights, in order to compete with other temporary flood defences which do not have the luxury to be engaged autonomously. This offers a promising opportunity. Furthermore, as it is required in some areas nowadays to preserve the townscape and maintain historical-cultural and natural values, new hydraulic structures are challenged to be integrated as much as possible in the area to the point that they are invisible if they are not closed. This brings up the question to design structures without support structures above ground surface level. This will have a major effect on the foundation.

Adaptivity

Most water retaining structures are designed for a lifetime of 100 years. For instance, the Self-Closing Flood Barrier^{TM} of Hyflo B.V. has a design lifetime of 100 years (Hyflo BV, 2021). However, boundary conditions can change in time. Extreme flood situations are occurring more often, even in the Netherlands as was described earlier in Chapter 1. Adaptive designing of flood protection structures can be of more importance nowadays than designing for a large lifetime. The goal of an adaptive design is to save the amount of materials by integrating a feature in the initial design that the design can be adjusted to comply to new boundary conditions rather than constructing a new structure. Functionally adapting a design should be within the end of the structural lifetime of the materials. Otherwise, the materials should be replaced anyways. With other words, an adaptive structure is designed with a shorter functional lifetime than the structural lifetime. This has an additional advantage, because designing for a longer lifetime goes hand in hand with more uncertainties regarding the hydraulic boundary conditions that are statistically predicted over that lifetime. Thus designing with adaptibility has the advantage of being more responsive to changing boundary conditions, which results in more accurately dimensioning which in turn contributes to material savings. This could be explored as one field of optimising a design of a self-closing flood barrier.

High purchase costs
In Figure F.2.4 a table is shown from the manufacturer of the Self-Closing Flood Barrier^{TM}, Hyflo BV. This exemplary table shows the results of a cost analysis considering numerous cost-related criteria, where three types of temporary flood barriers are compared, a movable, a demountable and a self-closing flood barrier:

Cost analysis of flood barriers	Mobile flood barriers	Demountable barrier	Self closing flood barrier
Purchase cost	xx	x	xxx
Training staff	xxx	xxx	0
Storage	xxx	xx	0
Transportation	xx	xxx	0
Maintenance	xxx	xx	x
Operational costs	xxx	xx	0
Operational staff training	xxx	xxx	0
Depreciation	x	xxx	x
Management costs	x	x	x

Figure 1.6: A cost analysis of three types of temporary flood barriers, (Hyflo BV, 2021)

In Figure F.2.4 can be seen that the purchase costs are scored the highest for a self-closing flood barrier in comparison with mobile or demountable flood barriers. This may be a serious reason for municipalities and provinces to not choose this type of structure. In order for this new promising type of temporary flood protection system to be implemented more in the Dutch water systems, it would be of interest to explore ways of optimising the design of the structure to reduce the manufacturing costs in terms of for example material use.

Autonomy
A self-closing flood barrier is usually designed with a drive mechanism that ensures an autonomous closure with an expected high water level by using buoyancy. This is not necessarily the only option to make a flood barrier self-closing. Buoyancy has its pros and cons and therefore it could be interesting to look into other drive mechanisms as a self-closing system.

Spatial integration
With changing boundary conditions, the demand for an increase in water retaining height in the coming decades is inevitable. For self-closing flood barriers, this will lead to possible issues with respect to spatial integration. This is especially the case for the barriers such as the 'vlotterkering' and the 'kleppenkering' since a larger retaining height is associated with a larger width in the open (out of use) state, hence more space usage and a larger obstacle free zone for the barrier. In highly populated and densely built areas this could lead to an infeasible situation, but perhaps also challenging to find a solution to integrate the structure nonetheless.

Also, given that larger dike segments may have to be designed for with a possible desire for self-closing flood barriers, connection of (multiple) individual closing gates will become a design problem, with respect to for example the water tightness or the coincidence with present structures. There is not always sufficient space, especially for a self-closing flood barrier to be integrated within densely built residential areas, touristic areas or other areas with high cultural, historical or economical value. Keeping as much buildings within the protection area of the flood defence section is a challenge, next to maintaining important access and evacuation routes and avoiding present foundations and cable and pipe works.

1.2.3. Problem statement

To summarize the complete problem analysis, self-closing flood barriers have an innovative way of mitigating flood risks, but may have opportunities for improvement, such as:

- Designing for protection heights higher than currently applied without intermediate supports above ground surface level lead to new challenges that require special attention to the structural design and in particular the foundation, which may direct in other solution directions.
- More consideration can be given to adaptivity in the design of the self-closing flood barrier, particularly in light of changing boundary conditions
- Investment costs are relatively high in comparison with other temporary flood barriers
- Current designs are autonomously self-closing with only the principle of buoyancy applied but may be of greater value if the system has other drive mechanisms ensuring this.
- The means of integrating a self-closing flood barrier in cities and villages will become more problematic with the need for longer spans of the flood defence and an increase of the required water retaining height, with respect to water tightness and the spatial quality considering the preservation of nature, social and cultural-historical values.

1.3. Objective and Scope

1.3.1. Objective

The objective of this graduation project is to develop a customized design of an adaptive self-closing flood wall in the Netherlands.

1.3.2. Scope

The design study in this report is done for a dike section in the upper river region in the Netherlands.

Self-closing in this thesis does not limit to buoyancy driven closures but also includes other closing systems, like sensor initiated drive mechanisms.

The design study on the feasibility of a self-closing flood barrier will not take into account cost-analyses and analyses on societal and political influences.

This final report will provide a spatial-functional and structural design for a self-closing flood barrier. The project will not be detailing the design in such a way that it provides construction drawings as to construct the structure in practice.

The focus of the structural design is on the permanent structure and not the temporary structure in the construction phase.

1.3.3. Study questions

Based on the objective, the following principal study question can be formulated:

- How does a conceptual design of an adaptive self-closing flood barrier for an upper river region in the Netherlands look like?

This project has the following subquestions:

- How does the length of single structures as part of the self-closing flood barrier relate to the failure probability requirement for a dike section, considering the failure mechanism ”height of water retaining structure” and how does this result in the assessment?
- How does the length effect relate to the failure probability requirement for a dike section, considering the failure mechanism ”reliable closure” and how does this result in the assessment?
- Which driving mechanisms (i.c.w. retaining wall) are suitable solutions that are in line with the failure probability requirement for a dike section for the reliability closure assessment of the structure?
- How can adaptivity be incorporated into the design for the retaining structure and the retaining design height, taking into account a reduced design lifetime for the retaining function? (note: this does not include the foundation)
- What does the integration and connection of different dike sections within a dike segment look like with minimal deterioration on spatial quality and an aim for zero houses being excluded from protected area by the dike (in Dutch: buitendijks)?
- How does the design process change, especially considering the hydraulic loads, if the design should also be made for a location with a different failure probability requirement and boundary conditions, and then what are the final changes in the design?

1.4. Approach and report outline

1.4.1. Approach

System analysis
The system analysis consists of the site analysis, the functional analysis and the stakeholders analysis.

Basis of design
In the basis of design the programme of requirements are stated to which the final design should comply to. Furthermore, criteria are presented which are used to evaluate potential concepts with respect to each other in a MCA. Lastly, the boundary conditions are given.

Spatial-Functional design

Selection project location
The specific location is determined which is potentially the best option to place the barrier resulting from the requirements, boundary conditions and evaluation criteria.

Selection barrier type
A preliminary selection is made from an inventory of possible barrier types based on the requirements and a multi-criteria analysis. The remaining options continue in the next selection procedure.

Selection barrier type with drive mechanism
A final selection is made of one high potential concept for the barrier consisting of a barrier type with a drive mechanism. This results from a second more elaborate multi-criteria analysis.

Functional design of barrier components
The functionality of the selected concept is elaborated by designing the required components per functional phase in order to comply to the functional requirements. This is done with the help of preliminary calculation and drawings.

Determining the main dimensions
The retaining height is determined with designing on the failure mechanism overtopping and/or overflow following the Design Guide Hydraulic Structures of Rijkswaterstaat. This is followed by determining the total height and the width.

Determining the failure probabilities for non-closure
Based on the method of the Design Guide of Hydraulic Structures of Rijkswaterstaat the failure probabilities for the components with respect to the failure mechanism non-closure are determined to which they should comply to. This is done with the help of a fault-tree analysis with the failure probability requirement at the top following from the Dutch Water Code.

Making the design adaptive
The design will be adjusted in order to provide the possibility to adapt the retaining height. This is based on the hydraulic boundary conditions which depend on the KNMI climate scenarios and the functional lifetime.

Integrating the barrier in the surroundings
The barrier is visualised in the project area with the help of drawings made in Revit Structure containing all the required components modelled with the determined dimensions and boundary conditions.

Structural design

Constructibility
The construction method, excavation technique, foundation method, transport and logistics and the construction sequence are described. As an aid conceptual drawing are used.

Determining loads
From the failure mechanisms, the hydraulic loads and permanent loads are determined per critical situation divided into the construction phase and the use phase.

Designing on stability
The vertical stability, horizontal stability, rotational stability, uplift and piping are verified using hand calculation following the Eurocode and the Design Guide Hydraulic Structures.

Gate design on strength
The gate is designed per main component on strength to resist all the loads following the Eurocode where the main verification is the check on the Von-Mises stress in the governing cross-sections.

Generalisation and discussion
The final design is generalised by describing the changes in the design choices when the structure were to be designed for other typical areas in the Netherlands such as coastal areas or lower river regions. The discussion is done on the project, the design process and design choices.

1.4.2. Report outline

The main text of this final thesis report will follow with six chapters.

Firstly, Chapter 2 will start the thesis with the system analysis. Subsequently, Chapter 3 will define a basis of the design. Next, in Chapter 4, the spatial-functional design phase follows. Chapter 5 will include the structural design. In Chapter 6, there will be elaborated on the generalisation and the discussion is given. Lastly in Chapter 7, the conclusions and recommendations are given.

2

System Analysis

The system analysis consists of the site analysis, the functional analysis and the stakeholders analysis. The site analysis provides information on the city of Arcen with its characteristics, shows the current dike segment and visualises the area for a possible new flood barrier with the help of maps. Furthermore, the functional analysis outlines the principal and preserving functions of the barrier from which the functional requirements are derived. Lastly, the stakeholders analysis outlines the stakeholders from which requirements and evaluation criteria are derived in the Basis of Design in Chapter 3.

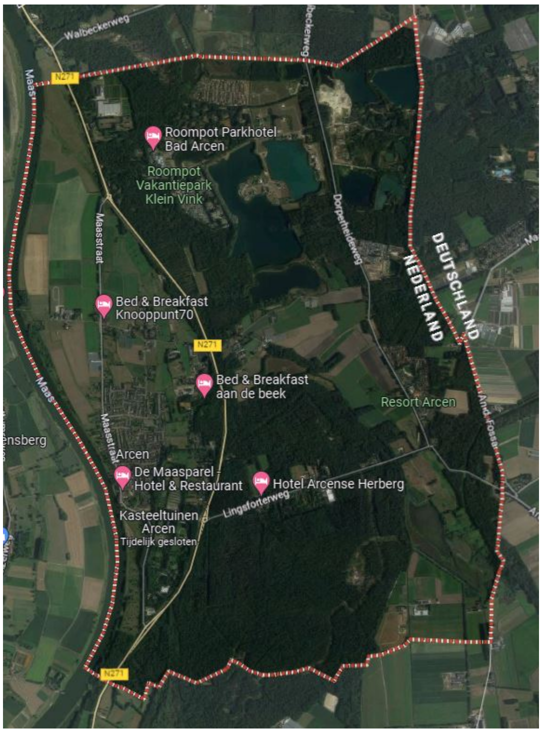
2.1. Site analysis

The project location is in a small village that is part of the municipality of Venlo in the south-eastern province of Limburg in The Netherlands, called Arcen. The village is relatively small with a population of around 2500 people and is situated in the north-east part of the province between the banks of the river Maas and the German border. This location provides nature areas with a rich variety of flora. Arcen is known for the nature reserves 'Maasduinen' and 'Barbara's Weerd'. 'Maasduinen' is a nature reserve which is bordered by the river Maas on the west side and a moraine on the north-east side across a region that Arcen is also part of in the southern part of the reserve. This protected natural area is characterized by dry heathlands with shifting river dunes. For this reason, it contributes highly to the ecology of the area and nature-oriented tourism. The 'Maasduinen' originated in the ice age by windblown river sand but the Meuse Valley now has height differences as the Meuse has embedded in over the years.

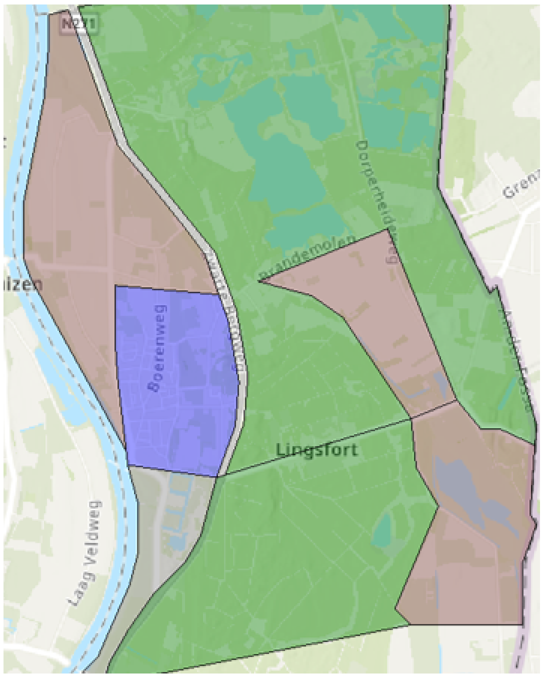
Moreover, the village has a valuable cultural heritage which includes the national monuments, watermill 'De Ijsvogel', the 'Schanstoren' and the historical castle Arcen with its castle gardens. Arcen is also known for its vacation resorts and Hertog Jan beer brewery and has for this reason a vivid tourism industry. The location close to the German border and along the river Meuse, provides easy access to other popular destinations in the region. The proximity of Arcen to the German border also makes it an attractive location for cross-border trade and commerce. In addition to this, the business activities in Arcen consists primarily of agriculture.

In Figure 2.1a a satellite view is shown of Arcen, in which the borderline is marked with red. As clearly can be seen, is that the village is located along the Meuse. Arcen can be subdivided into multiple area types. In Figure 2.1b a zoning plan is shown in which the different areas of Arcen are marked with a colour. Four types are discerned:

- Farmland
- Touristic and recreational areas
- Cultural-historical areas
- Residential/Business/Utility



(a) Satellite view of Arcen outlined with red (Google Maps, 2023)



(b) Zoning plan of Arcen (farmland in brown, touristic/recreational/nature in green, city-centre in blue, historical-cultural is grey)

Figure 2.1: Maps of Arcen

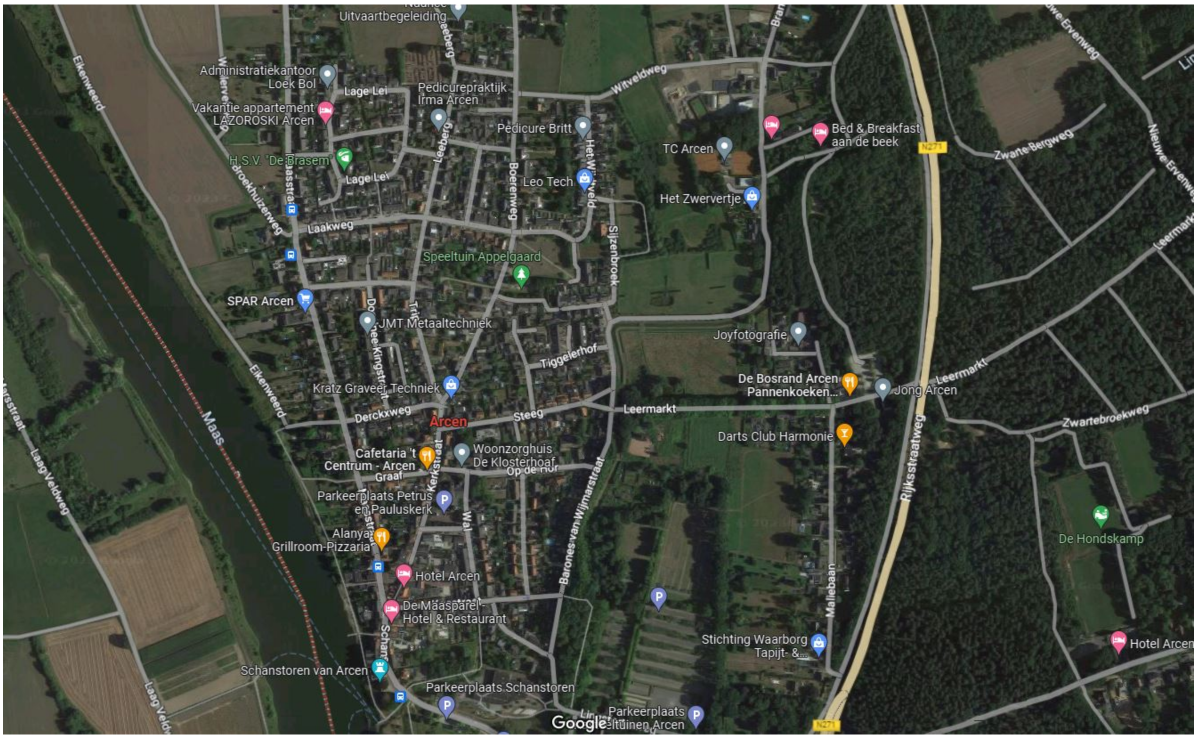


Figure 2.2: Satellite view of Arcen centre (Google Maps, 2023)

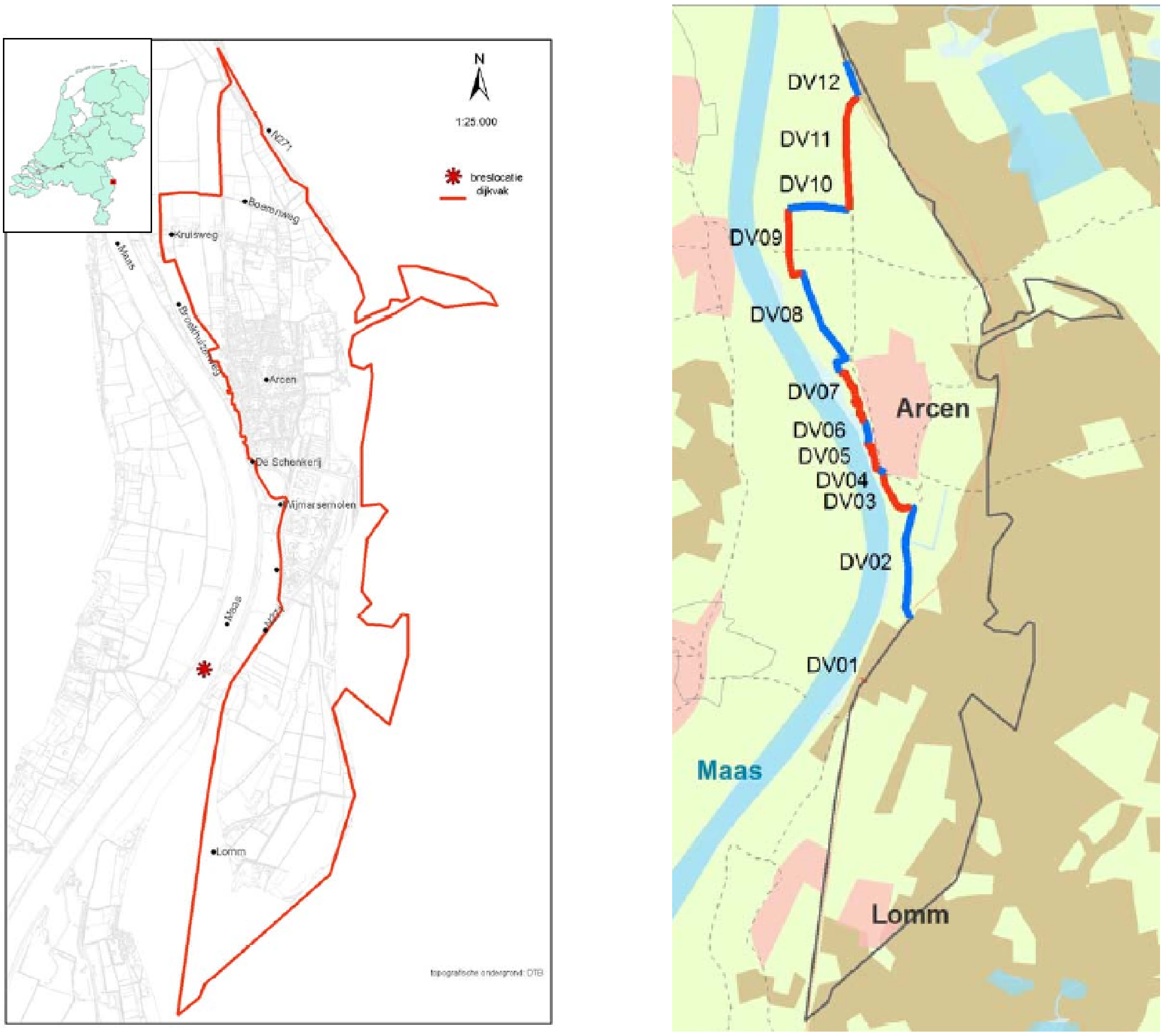
Figure 2.2 illustrates a zoomed in view on the city centre of Arcen. Most of the residential areas are clustered in this area. Arcen's remaining area consists of farmland and touristic areas such as resort parks, forested areas including a maze and a castle yard.

The majority of the houses and other buildings are enclosed by the Maasstraat and the provincial road N271. The Maasstraat is located near the Meuse. The Maasstraat begins approximately at the location of the two historical landmarks 'Schanstoren' and the 'Raadhuis' and has a length of approximately 2.5 km from which approximately 1 km is in the city centre. In the northern part, the street merges into the provincial road. The provincial road crosses throughout Arcen and connects the village with neighbouring villages. The provincial road clearly separates the residential area from the touristic and recreational areas.

Primary flood defences in Arcen

The National Primary Flood Defences Database (NBPW) describes the location and safety standards of primary flood defences in the Netherlands. These defences are subdivided into sections with a certain safety standard which are listed in the Dutch Water Act. Arcen also has a primary dike segment, marked with number 65-1. Figure 2.3a shows the dike segment. The dike segment covers the area of both Arcen and Lomm. In this site analysis only the part of the dike segment in Arcen is considered. Figure 2.3b shows the section layout of the section of dike segment 65-1 in Arcen. The current dike segment consists of twelve dike sections, comprising both engineering structures and grass dikes. As engineering structures, a retaining or quay wall (DV05 and DV07) or a temporary barrier are present, where the temporary barrier is a demountable barrier (DV06) or a temporary soil dam (DV02 and DV04). The line of the grass dikes (DV01, DV03, DV08 to DV12) are clearly visible in a relief map, shown in Figure 2.4 The current embankment at the location of DV05, DV06 and DV07 runs through gardens.

To the north, south and east of the dike segment, the function of flood defence is fulfilled by high grounds. The western part of the dike segment starts from the south with a dike section between Lomm and Arcen along Rijksstraatweg N271 through natural park 'Barbara's Weerd'. The embankment then runs along the west side of the castle gardens until pumping station 'Wijmarsemolen'. The barrier then continues westward from the 'Wijmarsemolen' pumping station to the 'Schanstoren'. From the 'Schanstoren' the barrier continues parallel to the 'Maasstraat' along the Meuse in the form of a quay or retaining wall until the end of the quay at Maasstraat 63 somewhere around the junction between 'Maasstraat' and 'Broekhuizerweg'. From this point, the embankment turns back into a grass dike that first runs along 'Broekhuizerweg', then continues around the 'Hertog Jan brewery towards 'Maasstraat' and continues north along provincial road N271 to the high grounds.



(a) Location of dike segment 65-1 and location designations. (VNK2: Overstromingsrisico dijktraject 65-1 Arcen, 2012)

(b) Section division of dike segment 65-1 (VNK2: Overstromingsrisico dijktraject 65-1 Arcen, 2012)

Figure 2.3: dike segment of Arcen

Arcen can be divided into three parts:

- Arcen North
- Arcen Centre
- Arcen South

Arcen North is the part with the grass dikes in the north, or dyke sections DV08 to DV12, in the area of the winter bed of the Meuse, where there are mainly agriculture and recreational areas and thus the areas with economic functions and landscape qualities. Arcen Centre is the section with the flood defence structures, such as the quay or demountable barriers (DV05 to DV07), mainly located around the town centre, where most of the residents live and where spatial quality, townscape and accessibility are important factors. The last section is Arcen South where the connection is with the southern high ground, where cultural-historical landmarks such as the castle gardens and the natural areas that are present are of high value.

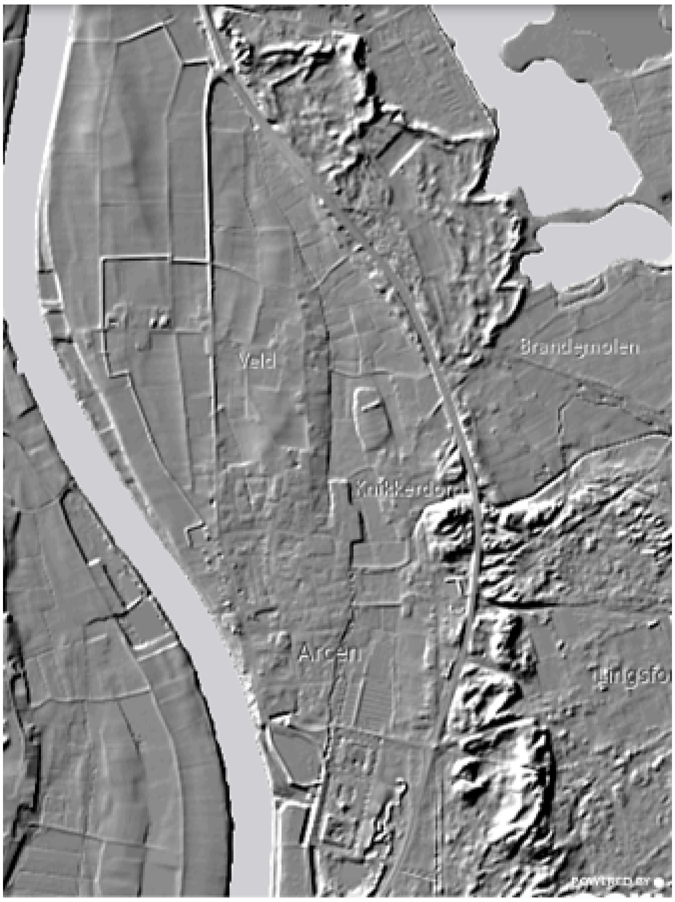


Figure 2.4: Digital terrain map of Arcen (AHN, 2023)

This site analysis shows that area Arcen-North and Arcen-South is not effectively suitable for a self-closing flood defence system because there are already grass dikes present that can be reinforced or relocated, which is more in keeping with the task of preserving spatial quality, nature, ecology and cultural-historical values, as well as in terms of river management, meaning preserving the winter bed. In particular, the latter ensures that a self-closing barrier is not appropriate because the winter bed belongs to the river's storage capacity and so the self-closing barrier will often have to be kept deliberately closed which is not in line with its function. These solutions are also more appropriate in terms of costs in relation to the economic value of the direct hinterland. In addition, a natural embankment has less disruptive and destructive properties for ecology than a self-closing barrier. Considering the remaining part of Arcen, namely Arcen-Centre, the primary flood defence already consists of a temporary flood defence, namely a demountable flood defence and a quay wall. Thus, for the selection of the final location for the self-closing barrier, this area is the most likely, especially as factors such as spatial quality, townscape and accessibility are of high value in this area. In the document for the dike improvement, system measure and brook restoration task for dike section 65-1 at Arcen, a self-closing flood barrier was also considered as an alternative, but was ultimately not preferred, as the cost difference was too great between a permanent glass structure and the self-closing barrier. In this report, however, the starting point is the application of a self-closing barrier as a flood defence, whereby a cost estimate will be taken into account, but will not be used as a basis for rejecting the application of a self-closing barrier. From this framework, the selection process for a project site for dike improvement with a self-closing barrier will be different from that in the Water Board Limburg's dike improvement memo. The specific site selection is done in Section 4.1 in the spatial-functional design.

2.2. Function analysis

Principal function of self-closing flood barrier

- The main function of the self-closing flood defence is to retain water to prevent flooding caused by high water levels in near water body's by closing of the hinterland, in order to offer protection when primary flood defences fail to protect the hinterland from flooding

Preserving function of self-closing flood barrier

- Integrate in the area such that:
  - The spatial quality is maintained, which includes preservation of the townscape, sight on the Meuse, social functions, social safety and living in the area
  - Protected natural and cultural-historical values are conserved
  - Evacuation routes in case of contingency are kept available
  - The ecology is undisrupted

2.3. Inventory of stakeholders

Public Service Providers

- Ministry of Infrastructure and Waterworks
- Rijkswaterstaat
- Provincial Executive
- Water Authority Limburg (Waterschap Limburg)

Private Service Providers

- Investor
- Contractor
- Suppliers
- Project management team

Core stakeholders

- Committee Environmental Impact Assessment
- Spatial quality team

Periphery stakeholders
This group of stakeholders is actually the group for whom the project is meant to be be built.

- Foundation for advocacy for residents along the river Meuse Arcen
- Other residents
- Business owners or farmers
- Community council

3
Basis of Design

3.1. Program of requirements

In this section, the self-closing flood barrier is hereafter referred to as 'Structure'.

Table 3.1: Site requirements for construction of self-closing flood barrier

Site requirements
Location assures zero buildings involving homes, commerce and hospitality industry or other commercial buildings being excluded from the protected area by Structure.
Location should have a minimum elevation of NAP + 13 m if the length of the dike section in question is less than 50 m, in order to assure a feasible and realistic height of Structure.
Location has no obstructions in the soil or does not coincide with present foundations of buildings already built.
Location has sufficient space to integrate Structure.
Location is available for construction of Structure and is approved by local authorities.
Costs for affecting spatial quality factors including important evacuation routes, gardens, protected nature reserves, historical-cultural buildings or monuments, archaeological sites and the ecology in the area, should be compensated for.
Location has necessary adjacent extra works, such as connecting roads and sufficient space to facilitate construction of Structure.

Table 3.2: Functional requirements for self-closing flood barrier

Functional requirements
Structure should be designed with an annual flood probability of 1:100 for the dike segment which is translated to stricter requirements per dike section per failure mechanism.*
Structure should be designed with KNMI climate scenario W+ for the end of the lifetime or with climate scenario G or a reduced lifetime if the structure is adaptable.
Structure has a functionality lifetime of 40 years.
Structure should be integrable in the area in existing structures, the subsoil or existing grass dikes.
Structure closes itself both manually activated and autonomously.
Structure has adaptability for the water retaining function.
Structure offers possibility to test, inspect and maintain the system.
Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures.
Structure has sufficient water retaining height*
Structure has a reliable closing system.*

Table 3.3: Structural requirements for self-closing flood barrier

Structural requirements
Structure is constructible.
Structural integrity: stability, piping, strength, dimensional stability.
Structural durability with respect to material decay/degrade/decomposition/corrosion.
Structural design has built-in adaptibility for functionality.
Foundation of Structure has a design lifetime of 100 years.
Structural deformations are within limits.

3.1.1. Failure probability per flood defence section

It is important to translate the flood probability standard to failure probability requirement per failure mechanism for a single flood defence section. This derivation is twofold, because two phenomena are involved here, namely:

- the length effect
- the interdependence between the failure mechanisms

This is also illustrated in Figure 3.1.

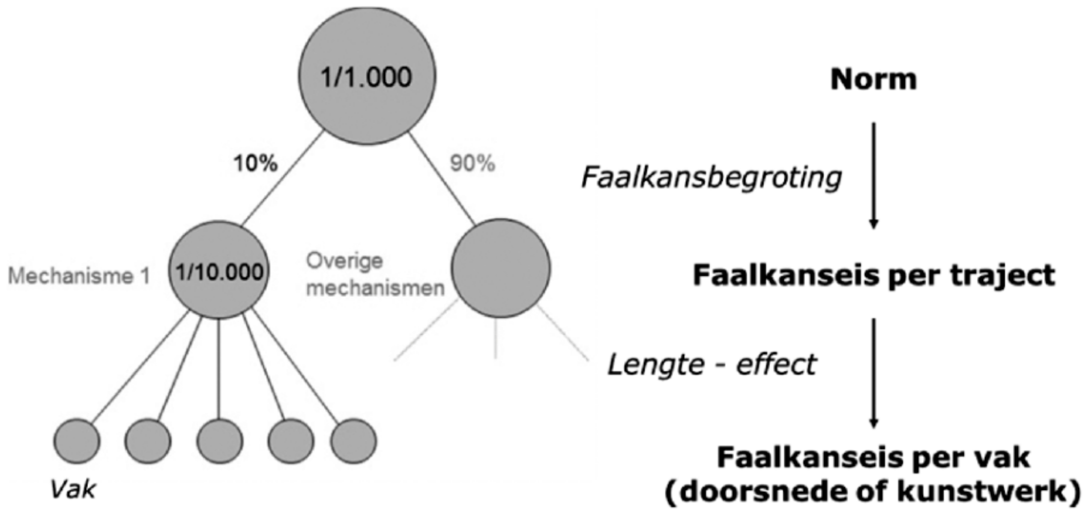


Figure 3.1: The determination of the failure probability requirement per section (section or structure) (Regeling veiligheid primaire waterkeringen, 2017)

The maximum permitted flood probability of the dike section (lower limit) per year is:

$P_{max} = \frac{1}{100} = 0.01[-]$

Interdependence failure mechanisms

In Figure 3.2 the failure probability estimates from WBI2017 and OI2014 can be seen which are stated by Rijkswaterstaat WVL (2017). For the dike section in Arcen applies the column 'Other', where the flood probability requirement for the dike section is divided among the relevant failure mechanisms.

Tabel 1. Default-faalkansbegroting uit het WBI2017 en het OI2014.

Type kering	**Faalmechanisme**	**Zandige kust**	**Overig (dijken)**
Dijk of kunstwerk	Overloop of golfoverslag	0%	24%
Dijk	Opbarsten en piping	0%	24%
	Macroinstabiliteit	0%	4%
	Beschadiging bekleding en erosie dijklichaam	0%	10%
Kunstwerk	Niet sluiten	0%*	4%
	Piping	0%*	2%
	Constructief falen	0%*	2%
Duin	Duinafslag	70%	0% (10%)**
Overig		30%	30% (20%)**
Totaal		**100%**	**100%**

Figure 3.2: Failure probability estimate ω per failure mechanism (Jongejan, 2013)

Table 3.4: The failure probability estimate per each failure mechanism

Failure mechanism	**Fraction of failure probability**
Overflow/wave overtopping	24%
Failure of closing	4%
Piping	2%
Structural failure	2%

The breakdown over the failure mechanisms that applies for a hydraulic structure in the dike section in Arcen is:

Length effect
A dike section can be subdivided into multiple segments in which the loads and strengths are stochastically homogeneous (Rijkswaterstaat WVL, 2017). Each dike segment within a dike section contributes to the failure probability like a serial system. Rijkswaterstaat WVL (2017) provides in the OI2014 and WBI2017 fixed length effect factors are provided per failure mechanism that compensates for this phenomenon.

The failure probability requirement per single structure is given by (Waterveiligheidsportaal, 2023):

$$P_{req,HS} = \frac{P_{req}}{N} = \frac{P_{max} \cdot \omega}{N} \tag{3.1}$$

where:

- $P_{req,HS}$ = Failure probability requirement per failure mechanism for a structure per year
- P_{req} = Failure probability requirement per failure mechanism for a dike section per year
- P_{max} = Maximum permitted flood probability of the dike section (lower limit) per year
- ω = Failure probability distribution factor for the failure mechanism in question
- N = Length effect factor for the considered failure mechanism.

From OI2014 the following length effect factors should be applied for the different failure mechanisms (Rijkswaterstaat WVL, 2017):

Table 3.5: The length effect factors for dike section 65 (Arcen) per each failure mechanism

Failure mechanism	**Length effect factor**
Overflow/wave overtopping	N = 1
Failure of closing	N = min ((n_{kw}) ; 10)
Piping	N = min ((n_{kw}) ; 10)
Structural failure	N = 3

From the information above, the failure probability requirement per failure mechanism for a structure per year is:

Table 3.6: The failure probability requirement per failure mechanism for a structure per year

Failure mechanism	$P_{req,HS}$
Overflow/wave overtopping	0,0024
Failure of closing	0,000067
Piping	0,000033
Structural failure	0,00047

Here it is preliminarily determined that n_{kw} = 6 for the failure mechanisms closure failure and piping.

3.2. Evaluation criteria

3.2.1. Location criteria

The locations will be graded based on the four following criteria.

- Disruption ecology
- Spatial quality
- Efficiency
- Length of dike stretch

Disruption ecology
The selection of the location could affect present nature and ecology in the area. The more area of the structure coincides with nature and ecology, the lower the score is given with regard to this criterion.

Spatial quality
The project location area is densely built which means that the structure, depending on the selected dike segment for the exact location, may cross home gardens, touristic areas or general properties of business owners and residents living along the Meuse. This means that during construction at such a location the residents will endure construction hindrance, such as nuisance, unavailability of the construction area etc. The space for the construction area will be more limited if the location is near residential buildings. Because the area is also owned by the residential owners, they are important stakeholders with whom good terms must be agreed upon making the construction process longer and more difficult. Furthermore, if were to be chosen for a location crossing properties, it would mean that part of that land would be unprotected in case of an extreme flood situation for which the structure is built. More specifically, this means that future development plans of the owners for that area will be affected. Besides construction, the structure needs to have yearly maintenance in order to ensure functionality. This means that the residents at least once a year are affected by the structure if the selected dike segment is at the location of the gardens or business properties.

According to the aforementioned explanation of spatial quality, the grading of the locations with respect to spatial quality is performed in a way that relatively less construction hindrance, effects of yearly maintenance and possible loss of land property leads to higher score and vice versa.

Efficiency
If less social challenges to consider are present to achieve the desired result, the efficiency increases. If the amount of stakeholders or social factors, to take into account, increase with a location, the efficiency decreases. For example, locating a structure at gardens, other land properties or touristic areas will raise the need for making arrangements with the stakeholders. If an alternative location could be chosen with less societal issues, the decision-making process goes more quickly. This could be at the cost of other factors, such as the length of the dike segment.

Length of dike section
The larger the stretch of the structure is required at a particular location of the dike section, the less preferable it is to choose for that location, because a longer stretch means higher costs and duration of construction.

The scoring method is described in Appendix D.

3.2.2. Criteria for barrier type

The scoring method for the criteria are described in Appendix D.

Adaptibility
The level of adaptivity of the self-closing barrier to increase the water retaining height determines the grading of the concept with the respect to this criterion. The easier an adaptive design could be made, the higher the score is given. For example, for an arched shape wall, it is more difficult to increase the retaining height than extending a flat wall. Also, if more components besides the retaining wall are affected by adapting the water retaining height, the structure is graded with a lower score regarding this criterion.

Integrability
The barrier will be integrated in the area and possibly in the surrounding structures. Each barrier type has a certain extent of integrability in the surrounding area and structures. The more it integrates nicely without impact on the area and without adjustments to the gate, the more preferable the barrier type is. If a certain barrier type requires additional measures to integrate it in the area in comparison with an other barrier type, the grading is done with respectively a lower score. Also, the amount of space occupation across the width determines the score regarding the integrability

Maintainability
The level of complexity of the structure determines the need for maintenance. The larger the amount of elements and connections, the more maintenance is required. This is also true if the structure type is generally larger in size. Furthermore, the access to perform inspection and maintenance plays a major role in this review.

Efficiency
The efficiency relates to the amount of material usage that is required and the functional simplicity of the structure and foundation. If a massive and heavy foundation is required the efficiency goes down and vice versa. The functional simplicity focuses also on the installation of the gate and the complexity of the structure.

Costs
Determination of the costs are highly inaccurate because of no good references for this specific project and because the case is highly unique. The costs are for this reason omitted in the evaluation.

3.2.3. Driving mechanism criteria

Operational reliability

The possibility of a back-up drive determines the reliability of the system. In case of failure, a back-up drive system could enable the system to work anyways. For each drive mechanism, the back-up drive is different. Furthermore, each drive mechanism has a different operation cycle. The operation cycles have different number of processes and each process has a certain simplicity or complexity to it, related to the number of components involved.

Complexity

Driving mechanisms have a certain complexity, in relation to the amount of space usage. For example, the need for machine rooms determine the amount of space usage. Also, the realisation of the drive mechanism determines the complexity. This relates to the ease of construction. For example, the more is needed to arrange the self-closing principle, or the number of mechanical parts that are involved in having the system to work, makes the drive mechanism more complex. Simplicity is preferred in the driving mechanism.

Maintainability

One driving mechanism require more maintenance than the other, because of different aspects. For example if water and steel are involved, it means that the components involved in the mechanism have to deal with corrosion and should be properly maintained. Movable steel elements also need regular lubrication and the more connections are present the more inspections should be performed. Besides these, access to the maintainable parts play a major role in the maintainability and the safety during inspection and maintenance.

Sustainability

This criterion refers to the risks of pollution of the environment and the amount of energy consumption. The higher the risks the lower the score for this criterion and vice versa.

Adaptibility

This criterion is already explained in the criteria for the retaining wall, see paragraph 3.2.2.

Integrability

This criterion is already explained in the criteria for the retaining wall, see paragraph D.3.2.

3.3. Boundary conditions

3.3.1. Ground surface elevations

The dike section along Arcen-Centre has a total length of 700 m. For multiple cross-sections along the primary flood defence in Arcen-Centre the ground surface elevations are shown in Appendix D. According to these diagrams from AHN (2024), it follows that the average ground surface elevation in the residential area along 'Maasstraat' is approximately NAP + 15 to 16 m (AHN, 2024). The river bed of the Meuse along the river bank is at NAP + 11 m (AHN, 2024). It can be seen in the diagrams that the surface elevation towards the river rapidly decreases. In the northern part of the dike section, the foreland has a width of approximately 30 m (AHN, 2024). Towards the south the foreland width decreases to 6 to 8 m (AHN, 2024).

3.3.2. Geotechnical conditions

Geological structure of the soil

The soil characteristics along the dike section in question are shown in Appendix D, in which CPT-graphs obtained from DINOloket (2022) are presented for several locations and also reconstructed soil profiles. In summary, the soil directly along the Meuse consists generally of silty clayey sand in the first 1 to 3 m (DINOloket, 2022). At some specific locations the soil consists of clay in the first 1 to 3 m (DINOloket, 2022). The soil below these layers in the area consist of medium to coarse sand layers to a depth of approximately 8 m + NAP and below the sand layers the soil consists of gravel only (DI-

NOloket, 2022).

Underground utility systems (KLIC)
These preconditions are disregarded in this thesis but do have importance in relation to the constructibility.

3.3.3. Hydraulic conditions

External water level and significant wave height
Rijkswaterstaat WVL (2021) provides the software Hydra-NL, that computes two preliminary hydraulic boundary conditions that will serve as the starting points for the design water level:

- The external water level
- The significant wave height

In Appendix J the input screens for Hydra-NL are shown. According to Rijkswaterstaat WVL (2018), the maximum permitted flood probability should be used for determining the hydraulic conditions in Hydra-NL, which is stated in the design guide for hydraulic structures (Werkwijzer Ontwerpen Waterkerende Kunstwerken). The maximum permitted flood probability follows from the Dutch Water Act and is denoted as a lower limit value which is $P_{max} = 0.01$ (see Section 3.1.1). The return period is then $f = \frac{1}{P_{max}} = \frac{1}{0.01} = 100$ years. In Figure 3.3 the results are shown in the table. For eight locations the external water level and the significant wave heights are calculated for years 2050 and 2100 for two climate scenario's that are described by the KNMI (2015). Explanation for the different climate scenario's can be found in Appendix J.

Lower Limit (maximum permitted failure probability) = 1:100

	Location							
	15	16	17	18	19	20	21	22
Water Level WL [m]	17,249	17,211	17,211	17,209	17,19	17,179	17,175	17,147
Significant Wave Height HS [m]	0,762	0,783	0,776	0,799	0,798	0,816	0,496	0,511

Figure 3.3: Results of design water level calculation for the year 2050 from Hydra-NL

Calculations were performed for both view 2050 and 2100 for both climate scenarios. From these, two extremes for the outer water level were obtained for the years 2065 and 2125 with the use of interpolation and extrapolation:

- for the year 2065 and KNMI2006W+ climate scenario: NAP + 17.5 m
- for the year 2125 and KNMI2006W+ climate scenario: NAP + 17.8 m

These values will be the initial starting point for the design water level. The initial water retaining height thus is NAP + 17.5 m for 2065 considering major climate changes. If were to be designed for year 2125 with major climate changes, the preliminary design water level would be NAP + 17.8 m. This is the preliminary boundary condition for the adaptibility requirement of the structure. The structure will be preliminary designed with a design water level of NAP + 17.5 m but will have the possibility to adapt its design so that the water retaining function easily can be adjusted to a new design water level of NAP + 17.8 m. Note that these boundary conditions water level will be adjusted during the verification to height of the retaining structure where overflow and overtopping will be taken into account.

Groundwater table
The location of the measurement of the groundwater table is not exactly at the project location, since the structure will be around the foreland along the Meuse, but the measurement location is the only one in Arcen (Waterschap Limburg, 2023). Thus, for this reason the ground water table will be assumed the same as the water level of the Meuse, since the project location of the dike sections are along the Meuse.

4

Spatial-Functional Design

4.1. Selection of project location

4.1.1. Suitable project locations

As already discussed in Section 2.1, the suitable area for the self-closing barrier is in Arcen-Centre. Arcen-Centre covers the area between the ”Schans” and the junction between Broekhuizerweg and Maasstraat. In Figure 4.1 a map of Arcen-Centre is shown in which these two landmarks are indicated that demarcate the area in question for the placement of the self-closing flood barrier. When zoomed in on the plan area, it can be noticed that the area can be subdivided into five areas with more or less the same characteristics. The dike segment will in this way also be subdivided into five dike sections, which can be seen in Figure 4.1. Per each dike section, two alternatives are available for the placement of the barrier. In Figure 4.1 each track for the potential placement of the barrier is marked with a yellow or red dotted line. The selection of the exact project location for the entire dike segment will be done by separately considering each dike section and verifying the possible locations against the site requirements. After that, one of the possible alternatives for that particular dike section will be selected by evaluation criteria.

4.1.2. Location selection per dike section

In Appendix E the verification to the requirements of the possible locations per dike section is shown. For dike section 1, two possibilities remained which are further evaluated. For dike sections 2 to 5 only one possible location remain after verifying them against the site requirements. An elaboration for each dike section is given in Appendix E. The result of the selection of the definitive project location for the entire segment is marked with the dotted yellow line in Figure 4.1. Also, the begin and the end of the entire dike segment and the current retaining structure are indicated in Figure 4.1.

Dike section 1
The location is at the walking trail in the foreland of the Meuse and not unnecessarily in the area crossing the private gardens.

Dike section 2
The location is in the area crossing the private gardens, because right along the river bank of the Meuse, the ground elevation is too low.

Dike section 3
The location for this dike section is at the walking trail right next to the ’Maasterras’ to prevent impact on the restaurant owner, but since it is a small area, the trail will be elevated in order for a feasible retaining height.

Dike section 4
The location is in the area next to the current retaining structure between private gardens and the touris-

tic promenade, because right along the river bank of the Meuse, the ground elevation is too low.

Dike section 5
The monument 'Schanstoren' will remain outside the flood protection area, because it is considered not feasible in this project to include the monument.

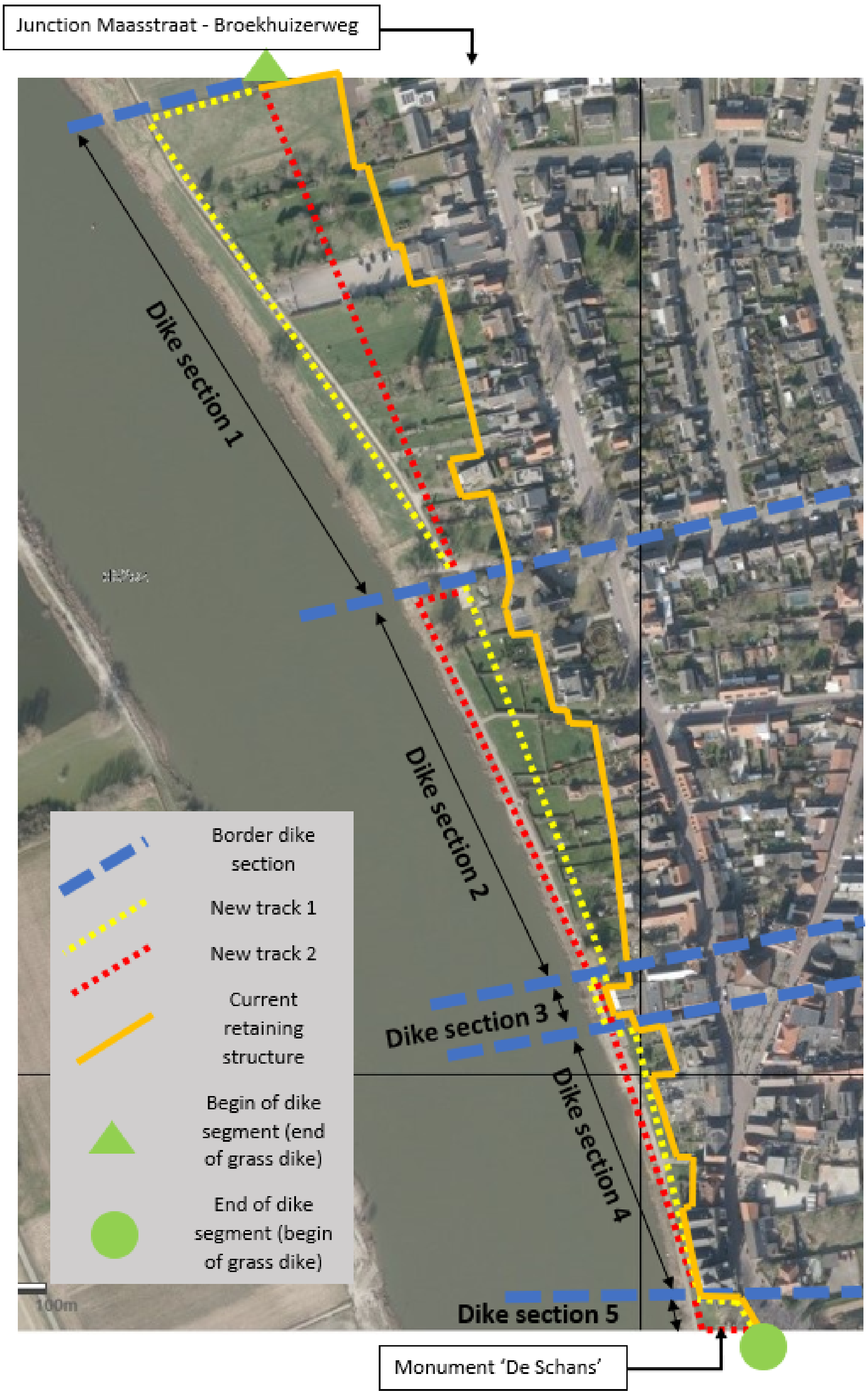


Figure 4.1: Project area with dike sections indicated (Figure obtained from Google Maps (2023))

In this report only dike section 2 will be further elaborated on with respect to the design of the self-closing flood barrier. This is because of the fact that it is the most interesting dike section because of the spatial quality over a large span i.e. limited space and affecting gardens and private properties of residents. For the selected locations for dike section 1, 3, 4 and 5, this is not a problem. However, dike section 3 is a special case because of the popular restaurant and the well-visited terrace and dike section 5 involves cultural-historical values that should be protected. In this thesis these aspects will not be focused on, because these sections involve small areas. Furthermore, similar to dike section 2, dike section 1 also has a significant length and large economical values behind the barrier. Nevertheless, for the location of dike section 1, potentially could be chosen to construct a grass dike instead of a self-closing flood barrier and in practice the placement of the barrier at this location would occupy a large area of the river's winter bed, which is not in line with a dike reinforcement plan in reality. For dike section 4, there is also limited space but does not involve the same amount of deterioration of spatial quality similar to dike section 2.

In conclusion, the design of the self-closing flood barrier will be made for dike section 2 and in particular for the track indicated with the yellow dashed line in Figure 4.1.

4.2. Preliminary selection barrier type

4.2.1. Inventory of existing barrier types

This section inventories existing concepts for movable water retaining barriers irrespective of the drive mechanisms, which can can be worked out further for the self-closing flood barrier. These concepts come already in various variants differing in materials and driving mechanisms.

The entirety of the movable retaining barrier and accompanying elements consists of (Erbisti, 2014):

- the leaf, which is the bulkhead i.e. dividing wall between the water retaining area and the hinterland. The leaf consists of:
  - skin plate
  - girders
  - seals for water tightness
  - support elements, such as wheels and rollers and guides

- embedded parts, which acts as housing and support for the leaf in order to redistribute the loads. The embedded parts are:
  - sill beam
  - wheel or slide tracks
  - guides
  - slot lining
  - seal seats

In Figure 4.8 a schematic overview is given in which different potential concepts are summarized based on various characteristics which can lead to the ultimate design. Horizontally moving barriers or barrier types rotating around a vertical axis are omitted beforehand, because without further review it can be concluded that these involve sight obstructing structures. Each concept is described shortly in the next sections. Appendix F provides more information on the gate types.

Flat gate (vertical translational)

As the type name indicates, this gate translates vertically to its retaining position. Dependent on the size of the structure and the retaining water height this structure has a relatively small width (Daniel & Paulus, 2018). The gates are relatively flat and are reinforced with rails and girders for extra stiffness if necessary. In this way larger spans are possible.

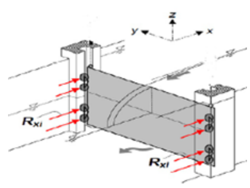


Figure 4.2: The principle of a flat gate for a spillway structure

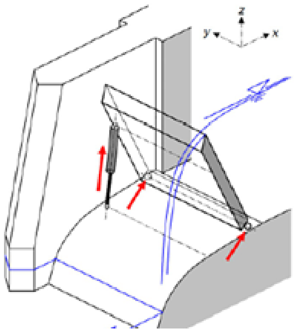


Figure 4.3: The principle of a flap gate for a spillway structure (Daniel & Paulus, 2018)

Flap gate
The flap gate hinges most of the time from a horizontal position (parallel to the surface underneath) to its retaining position. It requires an extra support to prevent overrotating (Daniel & Paulus, 2018). The flap gate hinges most of the time from a horizontal position (parallel to the surface underneath) to its retaining position. It requires more width than the retaining height.

Sector gate
The sector gate has the shape of a circle segment or a circle sector. The gate could be attached to a rotating disk or could have bearing arms which directly transfer the forces to the sill (Daniel & Paulus, 2018). A rotating disk or the bearing arms connected to a hinge allow the gate to rotate about the rotation centre at an angle at which the gate has a sufficient retaining height. Because of the arc shape of the gate, mostly normal forces are developed. The sector gate is in resting position integrated in the sill and therefore not visible. This type of structure is often used for locks and sluice gates to retain a large hydraulic gradient and is mainly executed with steel (Daniel & Paulus, 2018).

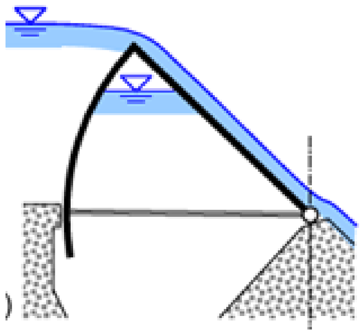


Figure 4.4: The principle of a sector gate for a spillway structure (Daniel & Paulus, 2018)

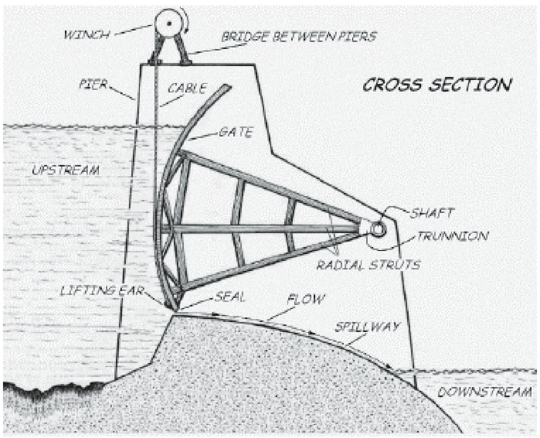


Figure 4.5: The principle of a radial gate for a spillway structure (Daniel & Paulus, 2018)

Radial gate
Only radial gates rotating about a horizontal axis are considered here. A radial gate has a arc shape and is essentially also flat. The gate can be reinforced with girders and posts and has bearing arms connected to a hinge (trunnions) allowing the gate to rotate (Daniel & Paulus, 2018). This type of structure is often used for locks and sluice gates to retain a large hydraulic gradient and is mainly executed with steel (Daniel & Paulus, 2018).

Visor gate
The visor gate is from top view visible as a circular arc and rotates around the horizontal axis (perpendicular to the flow direction) about the mid-line of the fictitious circle (Daniel & Paulus, 2018). It has the same principle as an eye visor of a Medieval helmet. Large spans are possible with this structure. This gate type is applied as part of the weir and lock complex near Amerongen in the Lower Rhine.

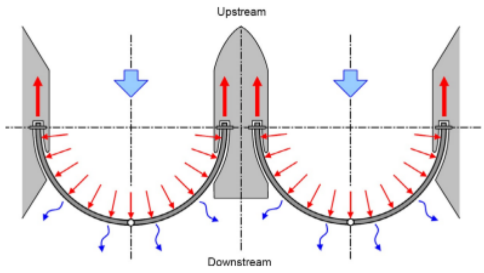


Figure 4.6: The principle of a visor gate for a spillway structure (Daniel & Paulus, 2018)

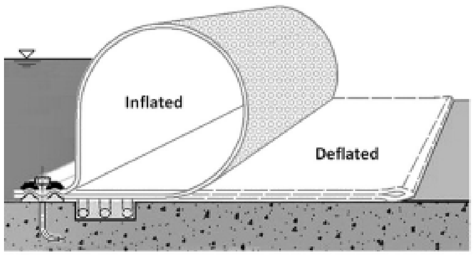


Figure 4.7: The principle of a bellows barrier for a spillway structure (Daniel & Paulus, 2018)

Bellows barrier
In the village Rampspol is a bellow dam constructed as a storm surge barrier which consists of a nylon and rubber fabric and inflates with water and air to become a barrier in case of a storm surge (Daniel & Paulus, 2018). Such a bellow dam can reach a water retaining height up to 10 m and has a corresponding width of approximately 15 m. The span for which it is applied is around 60 m to 80 m (Gebhardt, 2013).

Parachute barrier
A barrier type that is not used as of yet is the parachute barrier. The parachute barrier consists similarly to the bellow dam of a nylon and rubber fabric which can be clamped with robes and stretched by tensioning the robes in order for the parachute barrier to be deployed and reach its required water retaining height (Van der Ziel, 2010). The barrier only develops tensile forces and consists of relatively lightweight materials such as the bellow barrier (Van der Ziel, 2010).

4.2.2. Verification of barrier types to the functional requirements

The full verification procedure of the conceptual barrier types is given in Appendix G. The results are schematically given in Figure 4.8 below. As can be noticed, the visor gate, the bellows barrier and the parachute barrier do not comply to the requirements.

The visor gate does not comply to requirements:

- RQ-007: ”Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes.”
- RQ-008: ”Structure has sufficient water retaining height derived from RQ-001”

The visor gate requires to have piers above ground level in order to ensure the rotational movement around the horizontal axis. The structure does have the ability to be embedded in the subsoil at an angle to the horizontal. However, in retaining position at the supports (piers), the gate would not have enough retaining height, because of the orientation of the gate and the rotational movement.

The bellows barrier does not comply to requirements:

- RQ-003: "Structure should be integrable in the area in existing structures, the subsoil or existing grass dikes."
- RQ-007: "Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes."

The bellows barrier is as of yet not executed with a continuous span of 240 m. The possibility of such an application is still uncertain and might be infeasible. This implicates to look at already proven feasible solutions for this gate type which would be an application with multiple spans with bellows. However this results in the requirement to have intermediate support structures above ground level which is not in compliance with the requirements.

The parachute barrier does not comply to requirement:

- RQ-007: "Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes."

The parachute barrier requires pylons or similar structures to clamp the top of the fabric with ropes in order to stretch it so that sufficient tensile stresses develop. For this purpose, the pylons need to be above the ground surface level which does not comply to the requirements.

Conclusion of the verification
The visor gate, bellows barrier and a parachute appear not to be in compliance for a self-closing flood barrier in this case. This is summarised in Figure 4.8.

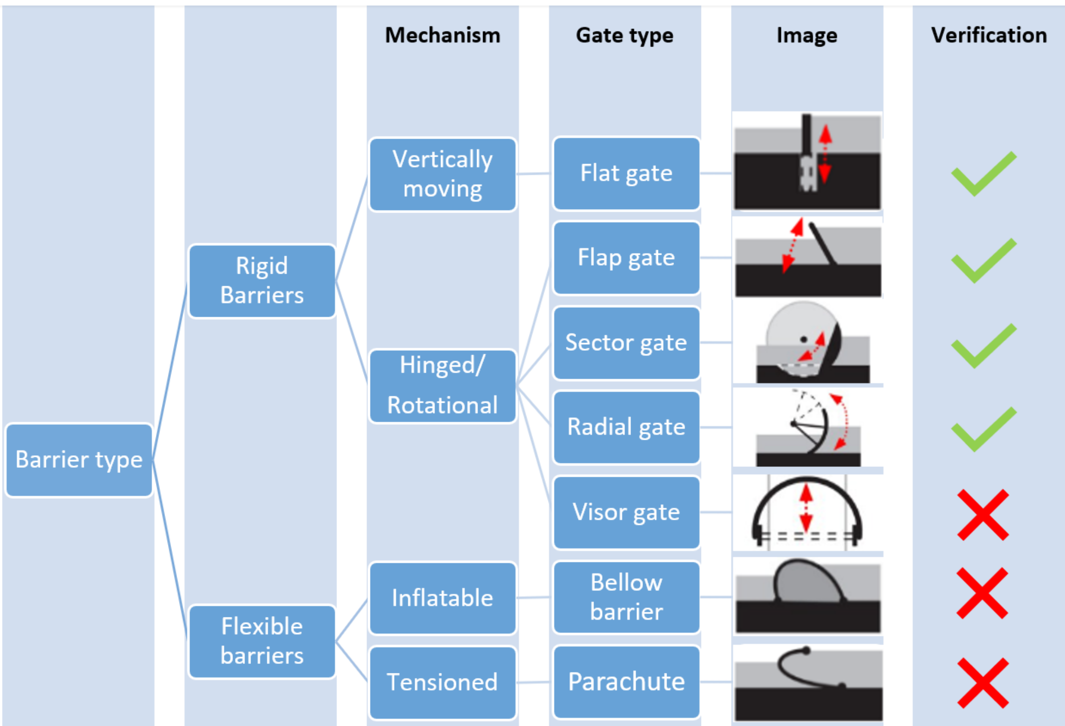


Figure 4.8: Schematic overview of verified concepts for movable water retaining barriers

4.2.3. Evaluation of barrier types

Allocation of weight factors to evaluation criteria

The determination of the weights given to each criterion is shown in Figure 4.9. If the criterion on the vertical axis is more important than the criterion on the upper horizontal axis, a '1' is given in the upper triangle of the matrix and a zero in the lower triangle in the diagonally mirrored cell. Each criterion receives a '1' with respect to its own. If the criteria are equally important, they both receive a '1'. In the end, for each criterion the total score is divided by the total points that are given to all criteria, which results in the weight factor used in the actual trade-off matrix.

	Adaptibility	Integrability	Efficiency	Maintainability	**Total**	**Weight**
Adaptibility	1	1	1	1	**4**	**0.36**
Integrability	1	1	1	1	**4**	**0.36**
Efficiency	0	0	1	1	**2**	**0.18**
Maintenance	0	0	0	1	**1**	**0.09**

Figure 4.9: Matrix to determine weights per criterion

Multi-criteria analysis

In Figure 4.10 the trade-off matrix is shown in which the gate types are graded based on the evaluation criteria from the basis of design. The justification for the scoring is given underneath the table.

		Sector	Radial	Flap	Flat	
Adaptibility	*0.36*	**1.0**	**1.0**	**3.0**	**4.0**	**[1]**
Efficiency	*0.18*	**2.5**	**2.0**	**3.5**	**3.0**	**[2]**
Functional simplicity		*2*	*1*	*4*	*4*	
Material usage		*3*	*3*	*3*	*2*	
Maintenance	*0.09*	**1.0**	**3.0**	**2.5**	**2.5**	**[3]**
Access		*1*	*3*	*4*	*1*	
Number of maintainable parts		*1*	*3*	*1*	*4*	
Integrability	*0.36*	**2.0**	**2.0**	**3.0**	**4.0**	**[4]**
Connection with area and surroundings		*1*	*1*	*3*	*4*	
Space usage		*3*	*3*	*3*	*4*	
Total		**1.6**	**1.7**	**3.0**	**3.6**	

Figure 4.10: Trade-off matrix for multi-criteria analysis

The flat gate scored the highest with **3.6**. Below the explanatory notes are found that justify the scoring for each gate type per criterion. The underlying reasoning can be found in Appendix G and detailed information about the gate types can be found in Appendix F.

1. Adaptibility
The sector and radial gate are scored with the lowest adaptability, mainly because adaptation to a curved skin plate is less straightforward than with a straight skin plate for a flap or flat gate. Between the flap and flat gate, the latter is considered less limited in its adaptibility in view of keeping a small obstacle free zone for the barrier, whereas an adapted flap gate increases in width and thus also the obstacle free zone. Nonetheless, adaptation to a flat gate results in deeper excavation and higher loads on the structure because of a lower foundation level, but here that does not outweigh the unfavourable effects on the spatial quality of the other gate types.

2. Efficiency
The material usage is scored based on the document 'Multifunctional movable flood barriers' written by Dijk and van der Ziel (2010) as part of a research to closures in the Rijnmond region for Royal HaskoningDHV. The report elaborates on a similar trade-off as this report but for dams and weirs in rivers. However, it still provides a way to indicatively say something about the material usage. Summarising, the flat gate uses generally more material than the other gate types. A reason for this is because it uses global bending to direct the loads towards the foundation, where the other gate types make more use of normal forces. On the other hand, for the functional simplicity, the flat gate is scored best because it does not involve a rotational movement like the other gate types, which generally is more complex than a linearly moving gate.

3. Maintenance
The flat gate is scored low with respect to the access for maintenance, since a flat gate generally has a narrow recess in the concrete structure. A sector gate is similarly scored low because of poor accessibility for maintenance, because the sector gate is stored in a self-containing recess in the sill (Dijk & van der Ziel, 2010), leaving little to no space. The flap gate and radial gate do not cope with the same problems. Furthermore, the number of maintainable parts is the highest for the gate types with a rotational closure movement, since these have hinged supports which need regular lubrication. The flat gate requires maintenance to for example guides, rollers and/or slides (Daniel & Paulus, 2018), but these are not located over the entire span.

4. Integrability
The sector, radial and flap gate require a space width as an obstacle free zone that is equal or more than the retaining height. This is more than that is the case for a flat gate. Contrarily, with respect to the connection with the area and surroundings, the flat gate generally consists of a deep slender foundation which is in need of a more specialised design than shallow and wide foundations. However, a slender foundation is less likely to coincide with present underground structures and surroundings. The radial gate and the sector, by contrast are generally heavily founded, which is also not preferable for constructing a flood barrier in private gardens. A flap gate has the advantage that the foundation can be very shallow, but does require a larger width which still may conflict with present underground structures.

Selection of potential barrier types
The next step is to conceptualise the best scored gate types with possible driving mechanisms and evaluate in a second trade-off step the potential concepts for the design of a self-closing flood barrier at the selected dike section in Arcen in order to select one preferential alternative. However, based on the performed MCA, already some reasoning can be done so as to narrow down the amount of potential concepts to have a more concise second evaluation step.

Thus that entails that the sector and radial gate, which received a total score below the 2.0 out of a maximum score of 5.0, are further excluded in the process. It is most likely that one of these gate types will not be the preferential alternative, regardless of the outcome of the yet to perform second evaluation including the drive mechanisms. For this reason, the selection process for a self-closing flood barrier will continue with solely the flat gate type and flap gate type.

4.3. Final selection barrier type combined with drive mechanism

4.3.1. Inventory of drive mechanism

Cylinder (hydraulic, electric, pneumatic)

Cylinders are used to produce a force in order to move an object in a linear motion. The force can be generated in various ways such as:

- Hydraulic
  - An incompressible fluid is pressurized which delivers the power to produce the force. This produces generally high forces.
- Electric
  - An electric motor generates a rotary motion which converts into a linear motion by a flexible connection or a worm gear transmission. This generally delivers a precise motion.
- Pneumatic
  - A gas is compressed that delivers the power to produce the force. This generally produces a high speed motion.

Thus, various options are available for a direct cylinder driven motion for a self-closing flood barrier, but for an hydraulic structure an hydraulic cylinder is more suitable because of the high force motion. For a vertical movement, an hydraulic cylinder is also the best option because of the maximum available stroke.

Gears

A vertical or rotary motion can be generated by gears. Gears in combination with a rack produce a linear motion. A movable barrier can be vertically moved in place by a rotating gear which is engaged in a rack and is connected to the movable wall. A rotary motion can be perpendicularly converted by a worm gear transmission (Daniel & Paulus, 2018). Generally the rotational movement of gears are generated by hydraulic or electric motors (Daniel & Paulus, 2018).

Wire rope

Wire ropes are steel cables which can be used to lift object. It uses the tensile force of a cable to lift an object, when getting wound on a axis which is generated by a motor for example (Daniel & Paulus, 2018). For lifting a flood barrier section, multiple drums are required dependent on the diameter of the steel cable. The steel cables can be connected to a lug plate that is fixed to the gate.

Buoyancy

Buoyancy is the phenomenon where the gravitational force of an object (partially) immersed in a fluid results in an upward force exerted by the fluid. The object floats when these forces are in equilibrium. In the case that a particular object is a flood wall floating in a certain water body or basin, an increase of the water level inside that basin would perform a vertical movement of that flood wall as well, because the buoyancy allows the flood wall to move along with the rising water level.

Inflation

For an inflatable system the Obermeyer gate can be considered, where a steel panel is raised by the inflation of a connected air bladder. This could also be a driving mechanism for other gate types designed as a self-closing flood barrier. The original system is bottom hinged, but if the inflation of the air bladder is bounded in horizontal direction, it would lead to inflation mainly in vertical direction, which results in a pure vertical motion for the gate.

4.3.2. Inventory of activation systems

The drive mechanisms involving cylinders, a cable winch and gears or racks could autonomously be activated with sensor signaling if the water level in the Meuse reaches a certain signal level. For the buoyancy mechanism an autonomous activation system would be immediate inundation with a pipe and/or basin system that fills up automatically, making the floating wall move up with the increasing water level. The Obermeyer gate system could be installed autonomously with for example a system similar to an airbag system or via a sensor.

4.3.3. Conceptualisation of barrier types with drive mechanisms

In this section suitable barrier types are combined with the inventoried drive mechanisms to create potential concepts for a self-closing flood gate system. For this conceptualisation, drafts will be presented for each combination of a barrier type with a drive mechanism. These drawings give more insight which becomes helpful to evaluate the concepts with respect to each other, which is discussed in the next section 4.3.4. The reason for conceptualising first is because it has little added value to evaluate drive mechanisms separately, without knowing anything about their application to the different gate types. This will also become clear in the evaluation, which scores on specific criteria that otherwise would not have sufficient justification underlying them.

Selected barrier types
The barrier types involved in this are a flat gate and a flap gate, which are the result of the preliminary selection process in Section 4.2. The flat gate is a vertical flood wall which moves in the vertical plane and the flap gate is a hinged flood wall which rotates in the vertical plane.

Selected drive mechanisms
For the drive mechanisms and the activation systems there are no requirements stated in this project because all of them are proven technologies for gate closures in practice. This design project does not focus on aspects for which some of these drive mechanisms would be characterised unsuitable, such as a requirement where for example zero emissions are allowed. However, the drive mechanisms do have their own strengths and vulnerabilities to which they can be evaluated with respect to each other based on several criteria.

The drive mechanisms involved in the conceptualisation are a wire rope system, a cylinder driven system, a buoyancy driven system and an inflatable system. This means that the drive mechanism with gears and/or racks are excluded. The reason for this is that its execution seems to be too extensive. For example, in order for a gate type to be driven in motion directly by gears, the gate leaf should be executed with a (partial) circular gear or a rack. Since gears are made out of steel, this means that the weight of the gate section increases with this type of drive motion. It makes the gate unnecessary complex and ineffective for the design problem. Gears are however used indirectly as drive transmissions for other mechanisms such as a cylinder, drive arm or wire rope with drum (Daniel & Paulus, 2018), which are already drive mechanisms considered on their own. Lightweight structures have the preference in this thesis. In conclusion, this means that the gears will not be considered further for any gate type in this thesis.

Drafting of concepts
Thus, combining the selected barrier types with each drive mechanism culminates in the following eight potential concepts for the second evaluation step. A full elaboration is given in Appendix H.

1. Flat gate driven by cylinder

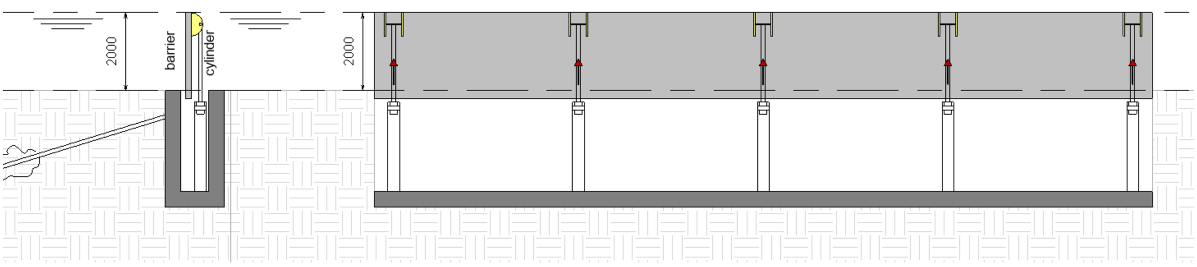


Figure 4.11: Draft of flat gate driven by cylinders in retaining position. Left: cross-section; Right: longitudinal section

2. Flat gate driven by wire ropes and drums

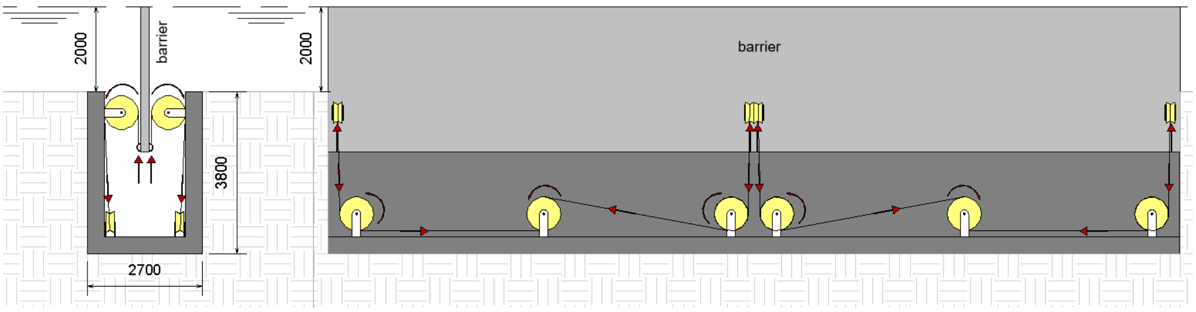


Figure 4.12: Draft of flat gate driven by a wire rope system in retaining position. Left: cross-section; Right: longitudinal section

3. Flat gate driven by buoyancy

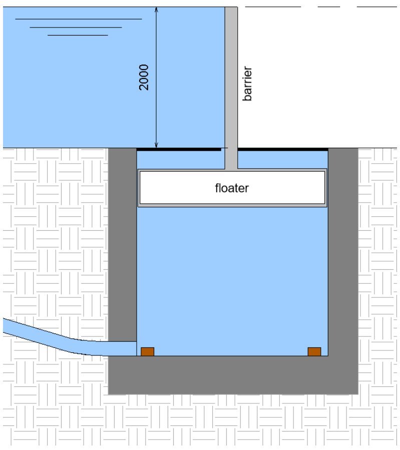


Figure 4.13: Draft of flat gate driven by buoyancy in retaining position

4. Flat gate driven by inflatable bellows

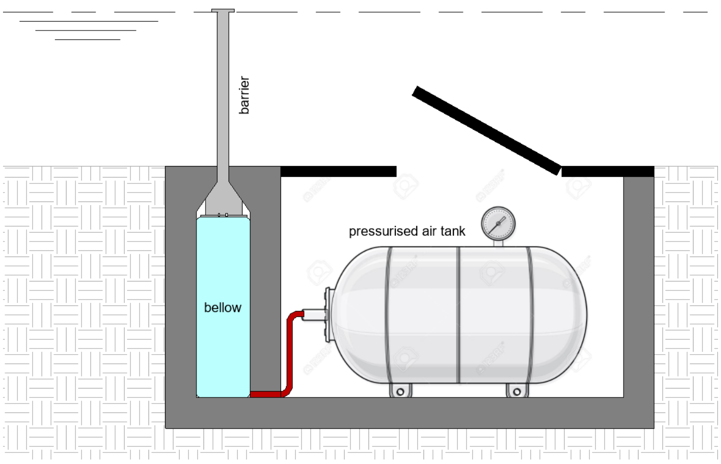


Figure 4.14: Draft of flat gate driven by inflation in retaining position.

5. Flap gate driven by cylinder

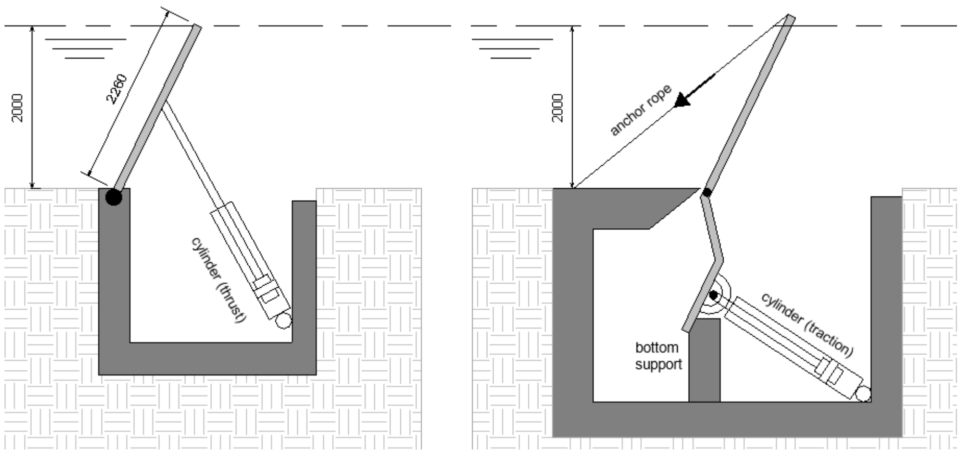


Figure 4.15: Two different drafts of one concept for flap gate driven by cylinders. Left: with cylinders in thrust; Right: with cylinders in traction

6. Flap gate driven by wire ropes and drums

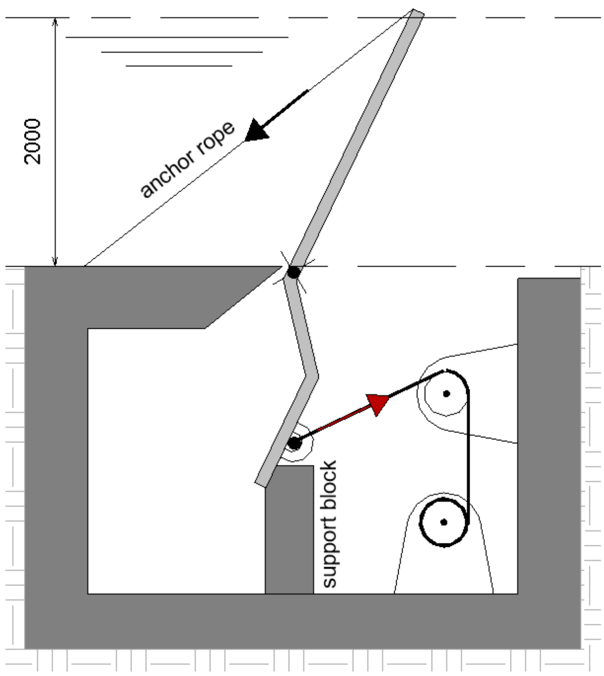


Figure 4.16: Draft of flap gate driven by a wire rope system

7. Flap gate driven by buoyancy

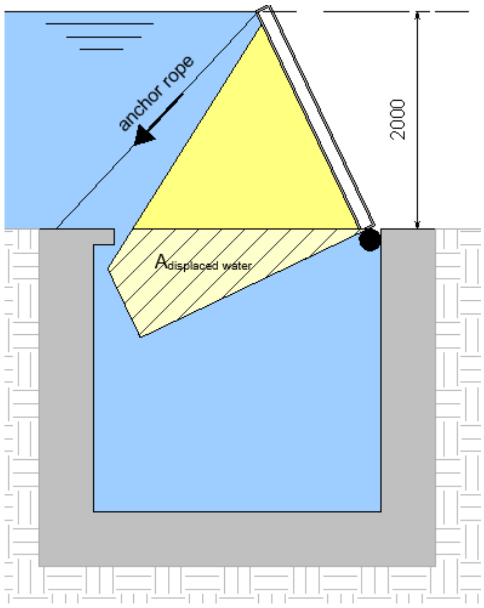


Figure 4.17: Draft of concept for flap gate driven by buoyancy with clockwise rotation

8. Flap gate driven by inflatable bellows

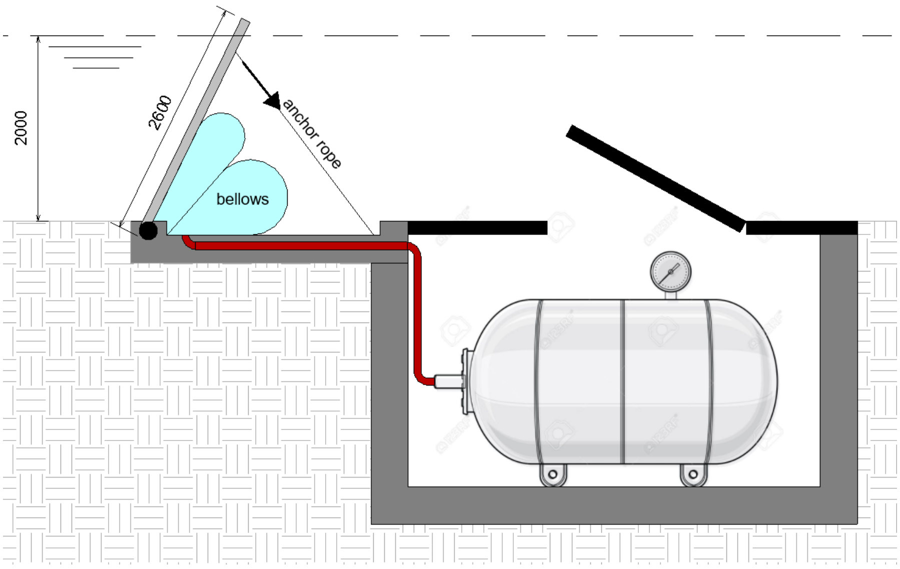


Figure 4.18: Draft of flap gate driven by inflation in retaining position

4.3.4. Evaluation of potential barrier types combined with drive mechanism

Allocation of weight factors to evaluation criteria

The determination of the weights given to each criterion is shown in Figure 4.19. If the criterion on the vertical axis is more important than the criterion on the upper horizontal axis, a '1' is given in the upper triangle of the matrix and a zero in the lower triangle in the diagonally mirrored cell. Each criterion receives a '1' with respect to its own. If the criteria are equally important, they both receive a '1'. In the end, for each criterion the total score is divided by the total points that are given to all criteria, which results in the weight factor.

	Reliability	Complexity	Maintenance	Sustainability	Integrability	Adaptibility	**Total**	**Weight**
Reliability	1	1	1	1	1	1	**6**	**0.23**
Complexity	0	1	1	1	1	1	**5**	**0.19**
Maintenance	0	1	1	1	1	0	**4**	**0.15**
Sustainability	0	0	0	1	0	0	**1**	**0.04**
Integrability	0	1	1	1	1	1	**5**	**0.19**
Adaptibility	0	1	1	1	1	1	**5**	**0.19**

Figure 4.19: Matrix for determining weight factors for MCA

Multi-criteria analysis (MCA)
In Figure 4.20 the trade-off matrix is shown in which the eight concepts of Section 4.3.3 are graded based on the evaluation criteria from the basis of design. The justification for the scoring is given underneath the table.

		Flat gate				**Flap gate**			
		Cylinder	**Wire rope**	**Buoyancy**	**Inflation**	**Cylinder**	**Wire rope**	**Buoyancy**	**Inflation**
Operational reliability	*0,23*	**3,5**	**3,0**	**4,5**	**2,0**	**3,5**	**3,0**	**4,5**	**2,0**
Back-up drive		*3*	*2*	*4*	*1*	*3*	*2*	*4*	*1*
Ease of operation		*4*	*4*	*5*	*3*	*4*	*4*	*5*	*3*
Complexity	*0,19*	**3,0**	**2,5**	**4,0**	**2,0**	**1,5**	**1,5**	**3,0**	**1,5**
Space usage		*4*	*4*	*3*	*2*	*2*	*2*	*2*	*1*
Ease of construction		*2*	*1*	*5*	*2*	*1*	*1*	*4*	*2*
Maintenance	*0,15*	**2,5**	**1,5**	**2,5**	**2,0**	**3,5**	**2,5**	**3,5**	**3,0**
Access for maintenance		*2*	*1*	*1*	*1*	*4*	*3*	*3*	*3*
Necessity of maintenance		*3*	*2*	*4*	*3*	*3*	*2*	*4*	*3*
Sustainability	*0,04*	**3,0**	**3,0**	**5,0**	**4,0**	**3,0**	**3,0**	**5,0**	**4,0**
Adaptibility	*0,19*	**4,0**	**4,0**	**4,0**	**4,0**	**3,0**	**3,0**	**3,0**	**3,0**
Integrability	*0,19*	**4,0**	**4,0**	**4,0**	**4,0**	**3,0**	**3,0**	**3,0**	**3,0**
Total		**3,4**	**3,1**	**3,9**	**2,8**	**2,9**	**2,6**	**3,5**	**2,5**

Figure 4.20: Trade-off matrix for scoring potential concepts including evaluation on the gate type with drive mechanism

Operational reliability
The operational reliability relates to the presence and the simplicity of the back-up drive and the operational ease. For a system using buoyancy as drive mechanism, there is no power source required and thus no need for a back-up unit. The recovery measure involves filling the floatation chamber with water through another way, such as with a water truck or finding the clog and unclogging it. Recovery measures do not involve relying on mechanical and electrical components, as is the case for the other mechanisms and it does not involve laboriously manually driving the system. Thus, buoyancy is scored the highest and the other drive mechanisms come next, with cylinders first in line, because of the option to have a damaged cylinder replaced in time before an expected high water event. Inflatable systems come as last because recovering a leakage in the bellows is almost impossible which immediately leads to external measures.

The operational ease for each drive mechanism is scored based on the number of processes needed to have the barrier closed. The more processes involved, the higher the risk of failure. For buoyancy, the operation is the most easy and for an inflatable system is the hardest also because of the additional aspect of uncontrollability of inflation.

Complexity
The complexity relates to space occupation of the gate with the associated drive mechanism and the construction ease. A flat gate scores higher in general with respect to the space usage. Besides that, there is a difference if machine rooms are involved or a pipe network for buoyancy. Generally, cylinders and wire rope systems need machine rooms and are thus the least efficient in space occupation. Bellows take more space in order to provide sufficient pressure to lift the gate and are thus relatively scores the lowest.

Generally speaking, flap gates need more support midspan against the risk of overrotation which has a downside on their end with respect to the construction ease. The number of mechanical parts needed for the drive mechanism to work also contributes to this. For a buoyancy system this leads to a good performance in the MCA and a relatively bad performance for a wire rope system.

Maintainability
The access for maintenance is undoubtedly better for a flap gate, because of the more spacious recess. Moreover, a flap gate driven by cylinders have good accessibility with respect to the other mechanisms because cylinders are generally accessible for maintenance in comparison to winched wire ropes, clogged pipes or bellows underneath a gate.

Evaluating based on the necessity for maintenance, leads to a different outcome, namely that a system driven by buoyancy requires less maintenance other than unclogging and cleaning of the pipe network if compared to drive mechanisms including mechanical components such as with a cylinder, wire rope or inflatable systems. However the difference is not major, because cylinders generally require little maintenance and the same holds for bellows.

Sustainability
Evaluating the sustainability of the drive mechanisms involve observing pollution risks and energy consumption for the system to work. For both aspects, buoyancy scores perfect because it uses water and hydrostatic energy. Cylinders and wire rope system however, do have pollution risks, related to hydraulic fluids and lubricants. But this does not hold for an inflatable system since that uses air. It does require external energy consumption, as is the case for cylinders and wire rope systems.

Adaptibility
The justification for scoring on adaptibility can be found in Section 4.2.3.

Integrability
The justification for scoring on integrability can be found in Section 4.2.3.

4.3.5. Selection of preferential alternative

In conclusion, the flat gate driven by buoyancy is scored the highest in the final evaluation step. This evaluation step involved a multi-criteria analysis for gate types combined with drive mechanisms, which can be seen in Figure 4.10. This means that for the self-closing flood barrier in this thesis, a flat gate with a buoyancy system will be designed. The following steps for the design involve firstly designing the barrier components on functionality and dimensioning of the barrier by verifying the height by an overtopping and/or overflow calculation. Then based on a verification related to the reliability of closing the barrier, failure probabilities are determined for the closing components. In the next section this is all further elaborated, together with functional aspects such as a closer look at the buoyancy driven closing mechanism, the functional adaptibility and integrality.

4.4. Functional design of the barrier components

This section elaborates on the functionality of the barrier which can be divided in four functional phases which respectively covers the open phase of the barrier when it is not in use, the closing phase of the barrier when it is being driven into its retaining position, the retaining phase and lastly, the re-opening phase. Each following subsection covers each functional phase which elaborates on the functional components or processes important for that particular phase. At the end of this section it should be clear how the self-closing flood barrier works.

4.4.1. Functional phase: barrier is open (not in use)

When the barrier is not in use, the barrier is 'open'. It means that it rests in the concrete structure which is embedded in the ground. Two important features are specified in this section which are the covering of the barrier and the support within the concrete structure.

Covering of barrier

In the open state, the barrier should not be visible and thus completely be integrating with the surrounding ground surface in terms of elevation. Furthermore, a potential opening should be covered in terms of safety. For this reason the concrete structure in the ground should have cover deck plates on top to cover the opening which might be visible. These cover deck plates can in turn be covered with for example (artificial) grass mats or wooden plates to blend in with the rest of the area of the gardens in which the barrier is placed.

The cover deck plates need to have supports over the entire span of the barrier since they can be loaded by humans or even be subjected by hydraulic loads on the river side. For the supports, steel girders with an HEA profile will be used which are attached to the concrete via an UNP profile which is bolted to the concrete. This needs to be modular, because the barrier requires the option to be lifted out of the embedded structure for maintenance and repair. This means that the steel console with cover deck plate needs to be detachable. The cover deck plate also needs to have a slight inclination in order to prevent accumulation of water when it rains. In Figure 4.21 and 4.22 impressions are given of the covering feature of the barrier.

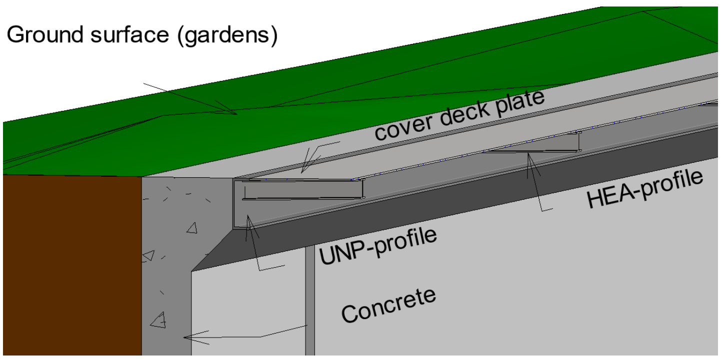


Figure 4.21: 3D impression of steel consoles with cover deck plate

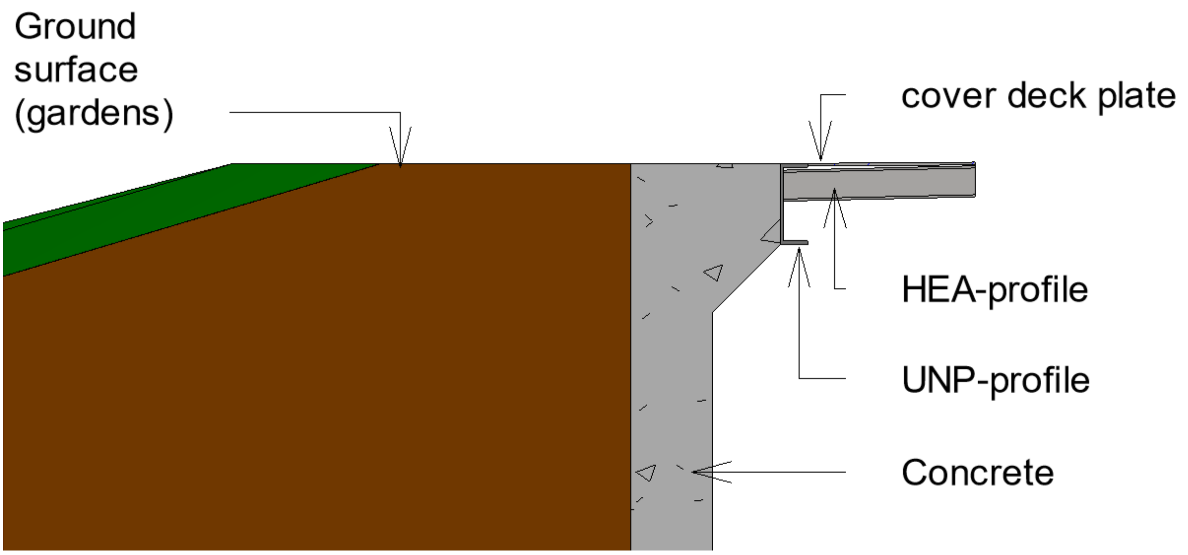


Figure 4.22: Cross-sectional overview of steel consoles with cover deck plate

From these impressions with proper engineering judgment it is assumed that an UNP300 profile will suffice. The structural verification of this profile is omitted in this thesis report.

Support blocks
In the open state, the barrier is supported on wooden blocks within the concrete structure. This is because the concrete structure will be designed for other hydraulic boundary conditions than the gate will be initially. This is related to the adaptibility feature of the barrier, which will be discussed further in Section 4.7. In short, the inner height of the concrete structure will be initially larger than the height of the barrier. The barrier will have the option to be increased with additional height. Initially, this is not the case and thus it is necessary that the gate rests on wooden support blocks so that the top the barrier properly aligns with the cover deck plates and the ground surface level.

4.4.2. Functional phase: closing of the barrier

Ensuring inflow and water storage
The barrier is closed by a floatation mechanism or a buoyant mechanism. For this to work a basin is needed which is able to fill up with water in which the barrier stays afloat. The concrete structure embedded in the ground will function as this basin and is called the floatation chamber. The floatation chamber needs inflow and outflow of water. For this reason several pipes will be connected between the floatation chamber and the Meuse. This requires additional excavation but also information on currently present cables and pipelines in the ground to determine where the inflow pipes should be located. However, this thesis does not further elaborate on this aspect. The level of application of the inflow pipes will determine at which signaling water level in the Meuse the system starts to fill with water. The outflow pipe is further discussed at the re-opening of the gate. At the end of the inflow pipe at the side of the river bank a certain water collection funnel can be attached in order to smoothen the inflow. At this collection basin a metal grid is connected to prevent inflow of debris which can clog the pipes. This water collection funnel needs regular maintenance to prevent clogging. In Figure 4.23 a hopper bin is shown which can be functionally used as such a water collection funnel. Also, the inflow pipe has a backflow valve to ensure flow in the pipe in one direction. This is because the inflow pipe solely should work as an inflow pipe to prevent outflow through the pipe because it can cause water hindrance at the promenade and cause erosion of the river bank. Furthermore, if the inflow pipes for some reason fail to fill the concrete chamber with water, a recovery measure is required. This will involve the call for a water truck that should provide a water volume of approximately 2900 m³ for the entire barrier. The underlying calculation for this is based on the dimensions of the structure which is covered in Section 4.5

Figure 4.23: A hopper bin, i.e. an example for a water collection funnel (Verachtert Nederland B.V, 2024)

Floating of the barrier

Objects can stay afloat in water because of a buoyant force. Buoyancy is based on the Archimedes' principle where an object that is partially or totally immersed in fluid is subjected by an upward force equal to the weight of the displaced fluid volume (Elger, Williams, & Crowe, 2013). When this buoyant force is equal to the weight of the object, the object stays afloat. The volume per unit meter span or cross-sectional area of the barrier which is initially assumed or related to an assumption for the weight of the barrier, needs to be 7.85 times smaller than the amount of displaced water. The reason for this is that the barrier is made from steel and steel has a volumic weight of 78.50 kN/m³ (NEN, 2019) whereas water has a volumic weight of 10.00 kN/m³ (NEN, 2019) which means that steel has a 7.85 larger weight which must be compensated for in the amount of displaced water for a floating object to be in equilibrium, hence be afloat. Thus, in order for the barrier to float, sufficient cross-sectional area must be provided which displaces the water. For this reason the barrier will be executed with an air tank, i.e. a floater on the bottom of the gate which provides the floatability. A floater can have many shapes but it also needs to ensure a floating stability. This is easier attainable by keeping the width to height ratio of the floater volume, i.e. the displaced water volume above 1. On top of this floater the flat retaining part of the barrier will be constructed. This leads to an upside-down T-profile for the overall cross section of the barrier.

Upward movement during filling process

In the following figures, Figure 4.24, Figure 4.25 and Figure 4.26 the filling process with the associated upward movement of the barrier is explained visually in three steps.

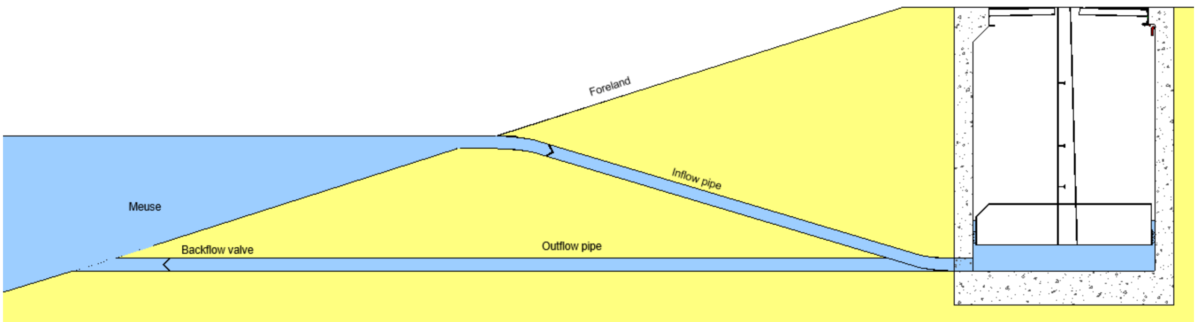


Figure 4.24: Step 1 of the filling process: the water level in the Meuse rises and the chamber fills with water

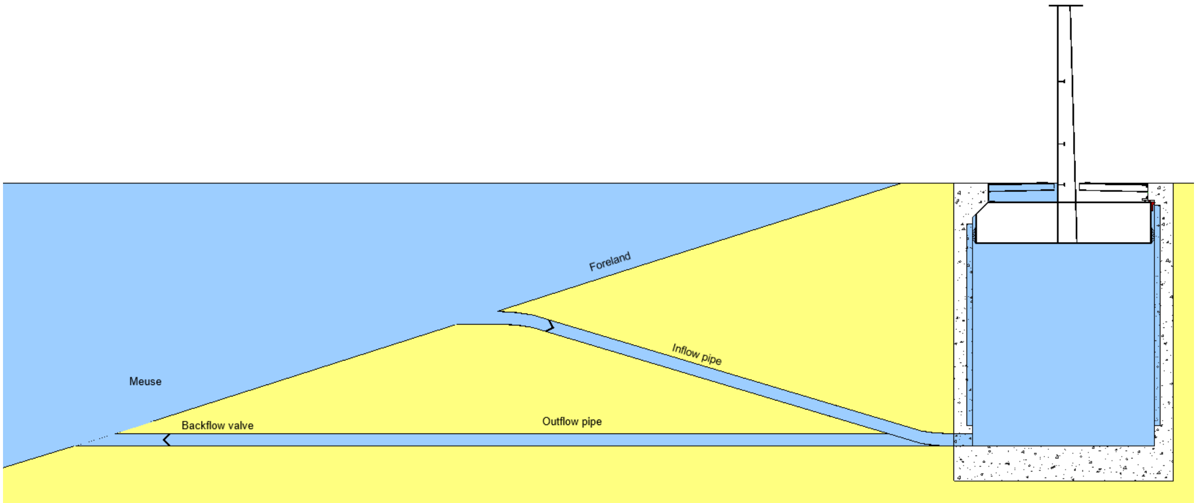


Figure 4.25: Step 2 of the filling process: the water level in the Meuse reaches the ground surface level and the chamber is completely filled with water, resulting in a fully emerged barrier

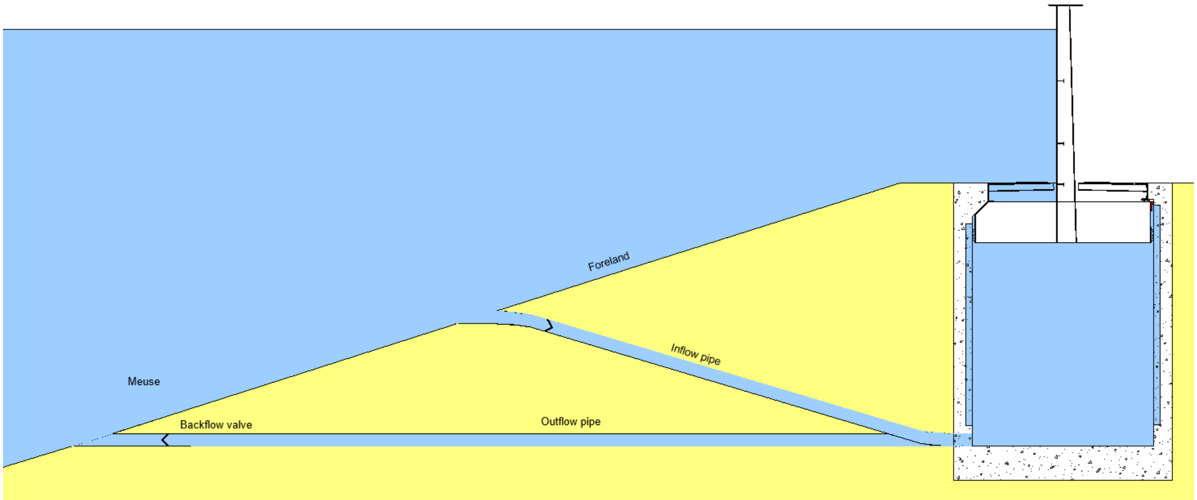


Figure 4.26: Step 3 of the filling process: the retaining water level increases and reaches the extreme level

Lateral guidance during upward movement

Along with the rising water level in the chamber, the barrier moves gradually upwards to its retaining level. During this upward movement of the barrier, lateral motion is possible by for example wind forces. This can lead to jamming of the barrier against the walls of the floatation chamber. To prevent this, the barrier is executed with four rollers at each side wall of the floatation chamber divided over the span and the concrete structure has guiding recesses embedded in the walls for the rollers to roll in. The rollers will help with guidance of the upward movement of the barrier. Rollers exert a certain friction force which actually counteracts upward movement of the gate (Erbisiti, 2014). For this reason, the floater should among other reasons be designed with additional buoyant force capacity in order to account for this counterforce. In Figure 4.27 a 3D section is illustrated of how the guidance feature is executed. In Figure 4.28 a cross-sectional overview is also given.

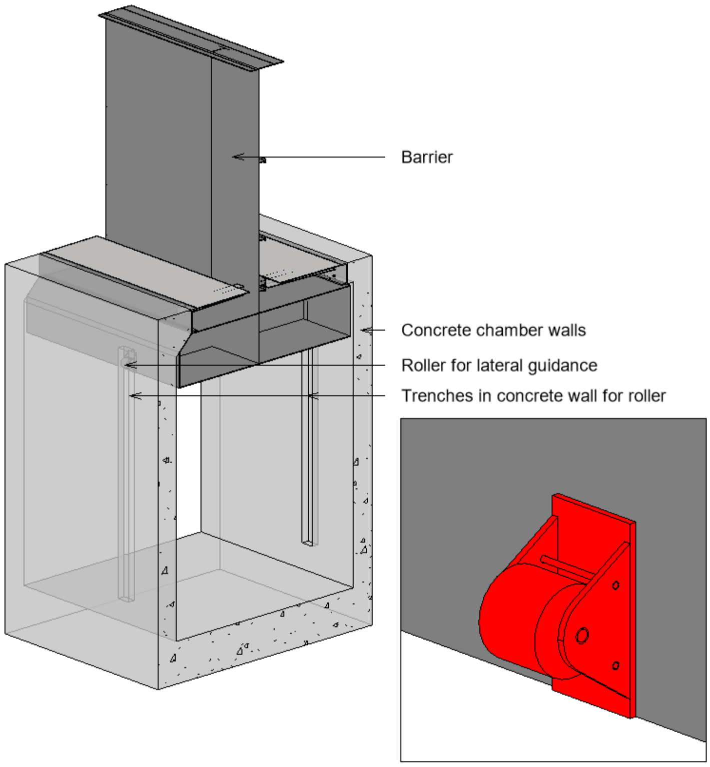


Figure 4.27: 3D model of lateral guidance feature

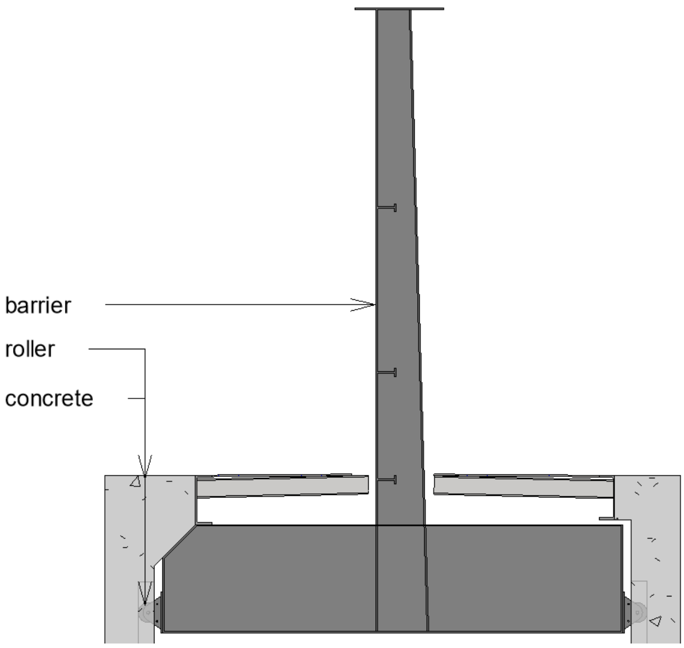


Figure 4.28: Cross-sectional overview of lateral guidance feature

Guidance in longitudinal direction during upward movement
During the upward movement of the barrier, also motion in longitudinal direction is possible. This can lead to clashing of the barriers against each other. To prevent this, the barrier is executed with two rollers at each side wall of the floatation chamber at midspan and the concrete structure has guiding recesses embedded in the walls for the rollers to roll in. The rollers will help with guidance of the up-

ward movement of the barrier and also prevent too large motion in longitudinal direction. However, in this way the barrier still has the ability to elongate at the ends of the barrier by for example temperature influences or straining because of bending. It is executed with tolerances in order to ensure free movability but with tolerances that are small enough to prevent jamming of the barrier in the recesses for the lateral guidance. In Figure 4.29 an enlarged picture of the roller structure is shown. In Figure 4.30 a top view is also given.

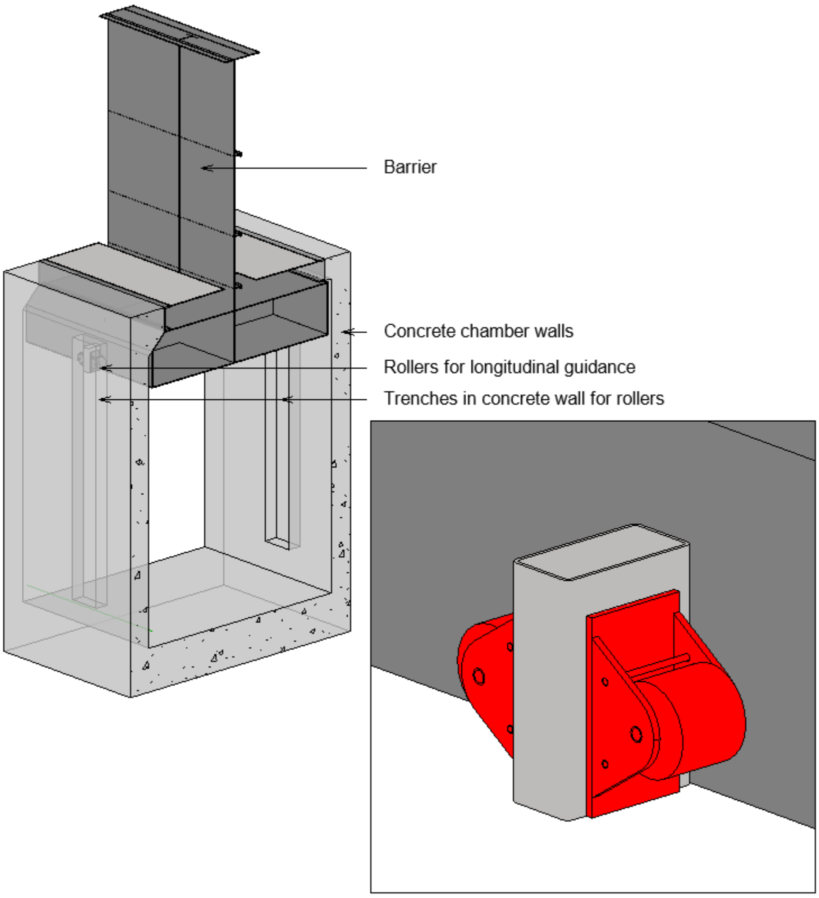


Figure 4.29: 3D model of longitudinal guidance feature

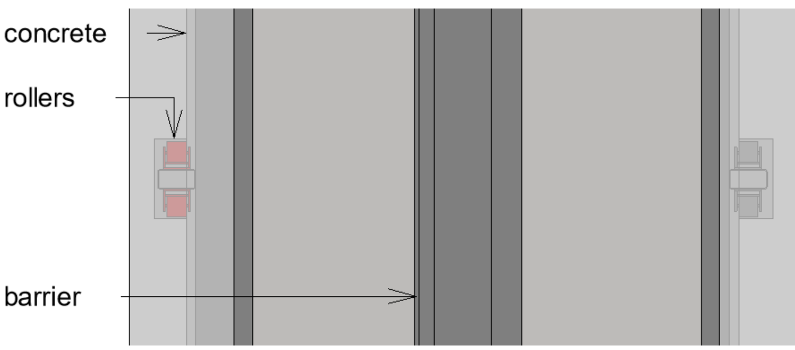


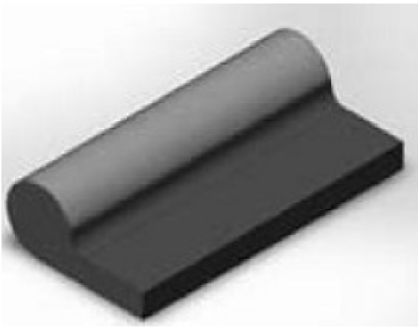
Figure 4.30: Top view of longitudinal guidance feature

4.4.3. Functional phase: retaining high water level

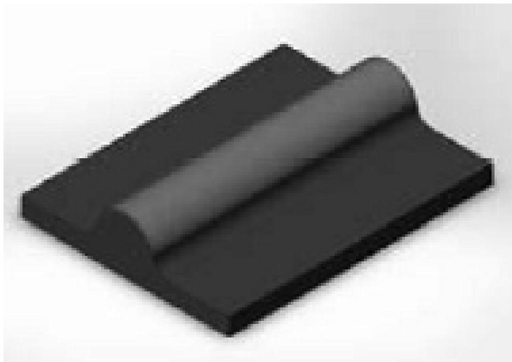
When the barrier reaches its top level and starts retaining the high water, there are two functional aspects to consider, namely the locking in of the barrier against the concrete in order to properly direct the forces to the concrete structure and secondly the sealing for water tightness in cross-sectional view but also in longitudinal direction.

Sealing for water tightness in cross-sectional direction

In cross-sectional direction the barrier needs to be sealed over the entire span on the protected land side, i.e. the dry side. The barrier is pointless if water still gushes out from the rear side of the retaining wall. For this reason two J-type seals are used, one with a single stem and one with a double stem, which are shown in Figure 4.31a and 4.31b. Seals are often executed with neoprene or natural rubber. However, neoprene has a greater hardness and is better resistant to weathering (Erbisti, 2014). Greater hardness is preferred for sliding seals which is the case in this design.



(a) Single stem J-type seal (Erbisti, 2014)



(b) double stem J-type seal (Erbisti, 2014)

Figure 4.31: J-type seals (Erbisti, 2014)

The spherical part of the seal is the part that is pressed between two connecting parts, allowing the part to deform and completely seal off an opening. The stem part is used to connect the seal to steel elements. The stem often has a seal clamp bar to keep it into place and to distribute loads that are being transferred between two elements over a larger area (Erbisti, 2014). This is illustrated in Figure 4.32.

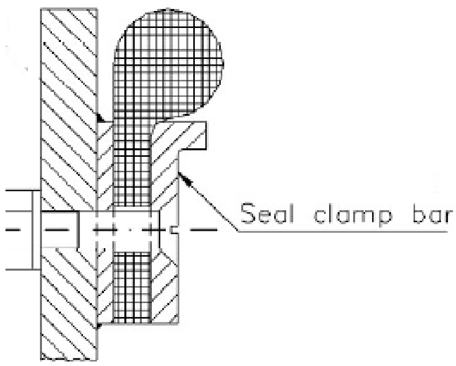


Figure 4.32: Example of connection with seal (Erbisti, 2014)

The application of the seals in the structure is shown in Figure 4.33. The seals are fastened with countersunk bolts and clamp bars. When the barrier is loaded, the seals are being pressed, resulting in deformed shapes of the seals. This is illustrated in Figure 4.34. This allows the barrier to transfer the loads over a specific area towards the concrete.

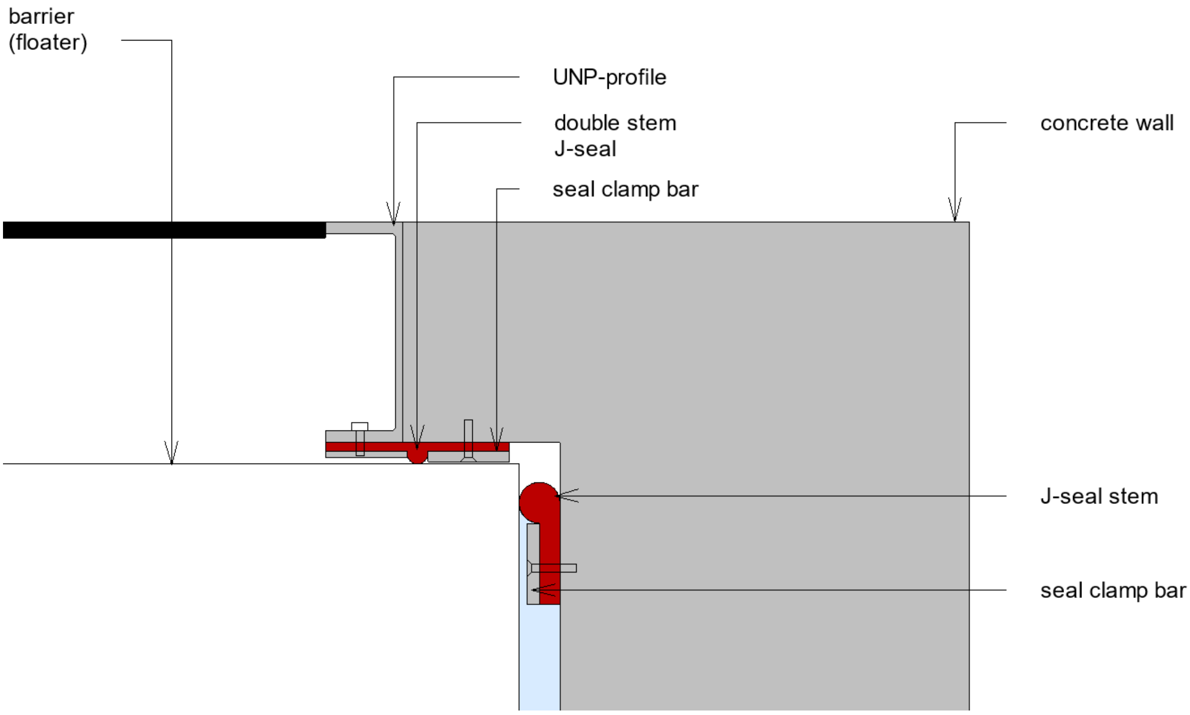


Figure 4.33: Illustration of the application of seals in structure at the dry (back side) of the barrier for water tightness in cross-sectional direction

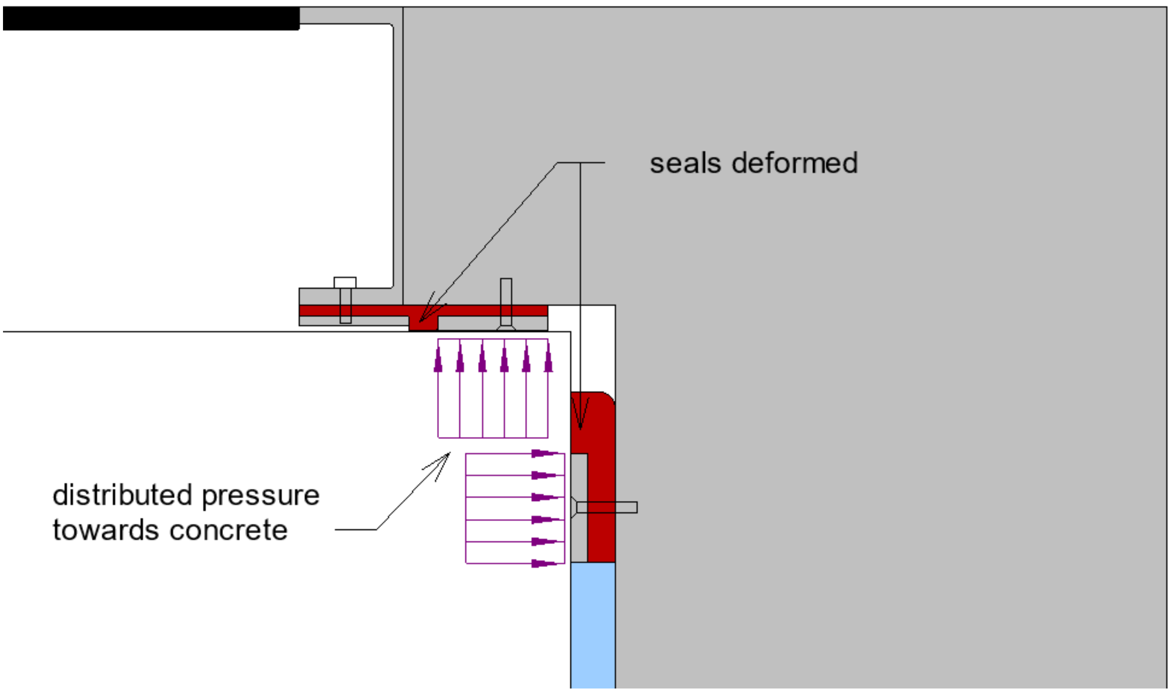
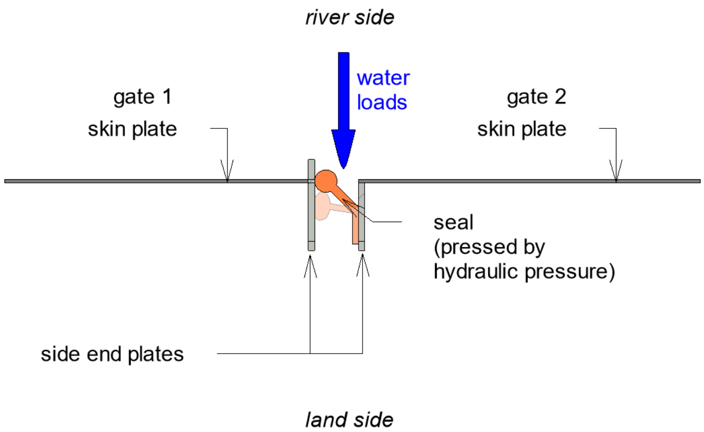


Figure 4.34: Illustration of activation of the seal during loading of the barrier where pressure is exerted on the seal leading to deformation

Sealing for water tightness in longitudinal direction
In longitudinal direction the barrier must be sealed in between two individual closing gates. Between the gates there will be a small gap. The gap is located across the entire height of the gates, which means that a seal across the height is needed which eventually connects to the seals for water tightness in cross-sectional direction. Angle shaped seals will be used for this purpose, which are shown in Figure 4.35a and will be attached to a side end plate on one side of the barrier. The connecting side of the following barrier will have a finished side plate in order for the seal from the previous barrier to smoothly move against. This is illustrated in Figure 4.35b.

(a) Angle shaped seal (Erbisti, 2014)



(b) Top view of sealing between gates: sealing gets automatically activated when hydraulic loads press against it

Figure 4.35: Overview of angle shaped seal

In Figure 4.36 and 4.37 the application of the seal for water tightness in longitudinal direction is shown. Important to notice herein is that the two seals for water tightness in different directions meet at one point in order to close off the entire gap. For this purpose the seal for longitudinal water tightness (indicated in orange) is configured in a L-shape.

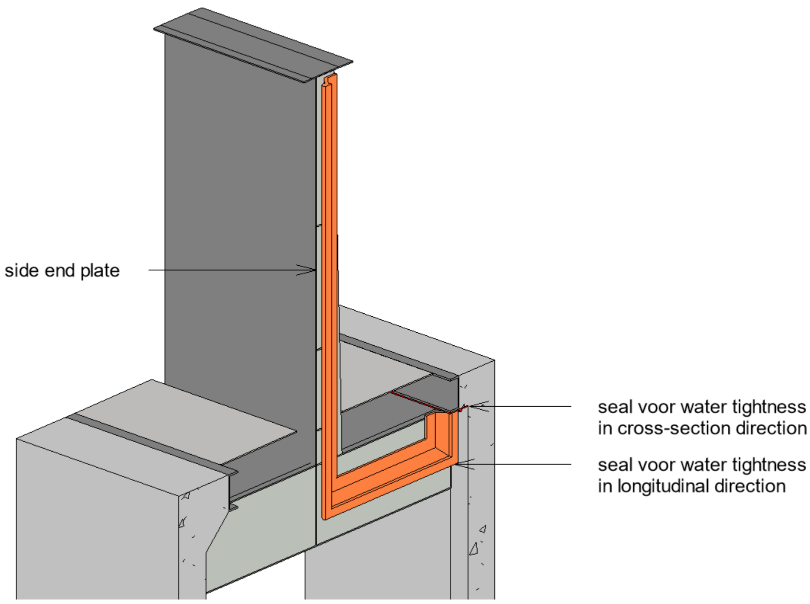


Figure 4.36: 3D impression of seal: the configuration over the height is L-shaped in order to connect it to the seal in other direction

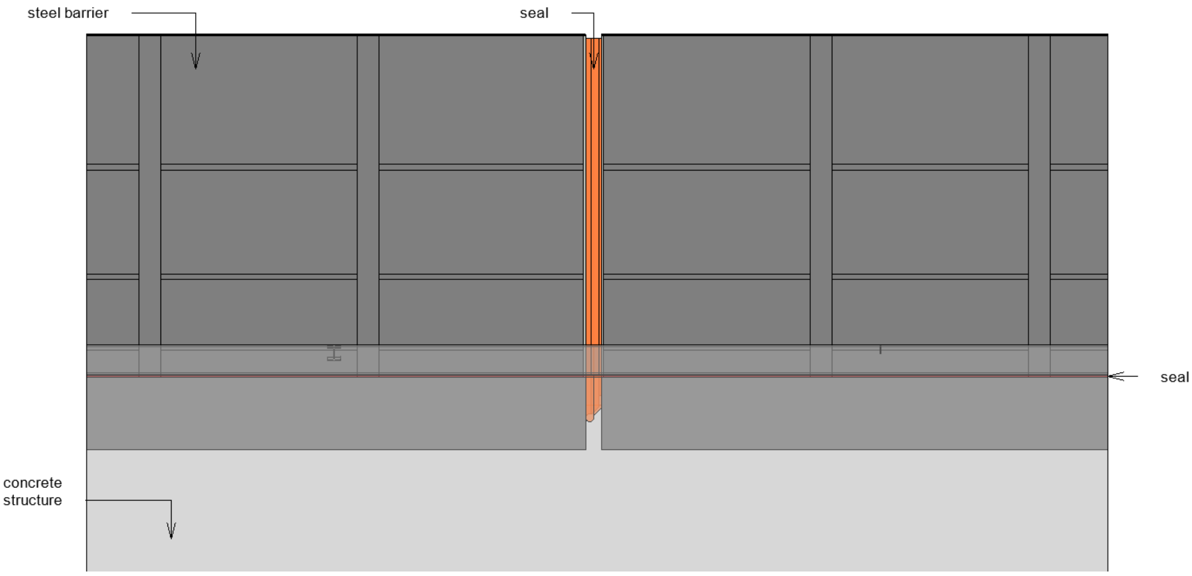


Figure 4.37: Rear view barrier with indication of seal between two gates

Locking in of the barrier against the concrete
The top of the concrete wall is executed with kind of concrete consoles. The steel consoles consist of the UNP300 profiles and the HEA profiles welded within for support of the cover deck. When the barrier is loaded by the hydraulic loads, the barrier is pressed against the seals which activates them. The barrier makes contact with the seal clamp bar of the seals ensuring automatically locking the barrier against the concrete. The equilibrium of forces on the barrier is elaborated in the structural design where the barrier is modelled in a free body diagram in which the support reactions that are distributed to the concrete are calculated.

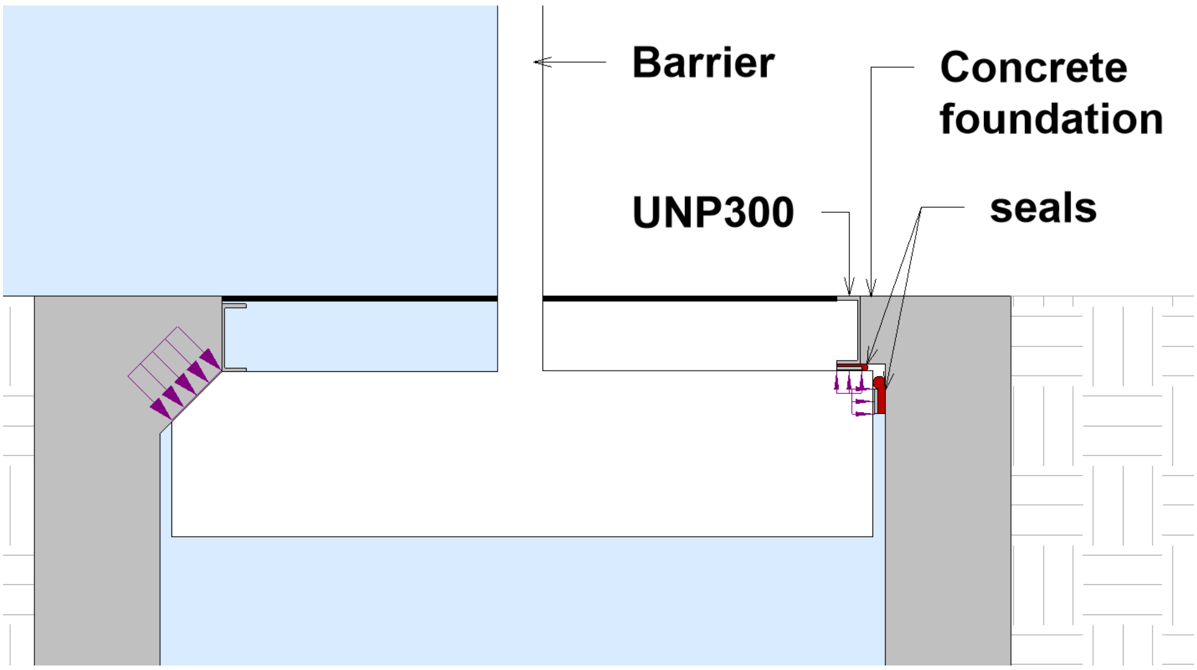


Figure 4.38: Cross-sectional view of locking in of the barrier

4.4.4. Functional phase: re-opening of the barrier

When peak levels of the extreme high water event have surpassed, the water level in the Meuse starts to drop gradually. When the water level reaches below the ground surface level, the concrete chamber, still filled with water starts to release water via the outflow pipes. The water level lowering results in re-opening of the barrier. The outflow pipes are executed with a backflow valve in order to ensure flow in the outflow pipe in only one direction, which is the backflow direction. Outflow via the inflow pipe is not desired as was mentioned earlier, which is why these are also executed with a backflow valve to ensure solely inflow. The outflow pipe needs to be connected to the bottom of the floatation chamber to prevent water from remaining in the chamber. After the concrete chamber is fully emptied and the barrier thus fully re-opened, it is time to perform maintenance.

4.5. Determining of the main dimensions

The main dimensions are determined in this section and involve the retaining height and the height and width of the floating body attached to the barrier. The retaining height is determined with the help of the software Hydra-NL, developed by Rijkswaterstaat, Deltares and HKV Lijn in water (2021), using statistics and prediction models based on climate scenarios that are formulated by the KNMI (2015). The dimensions of the floater are determined based on assumptions and an observation of the static floating stability.

4.5.1. Determining the retaining height of the barrier

The self-closing flood barrier requires sufficient retaining height in order to keep the amount of wave overtopping and/or overflow within acceptable limits to prevent flooding with substantial consequences. The fault tree for this failure process is given in Figure 4.39.

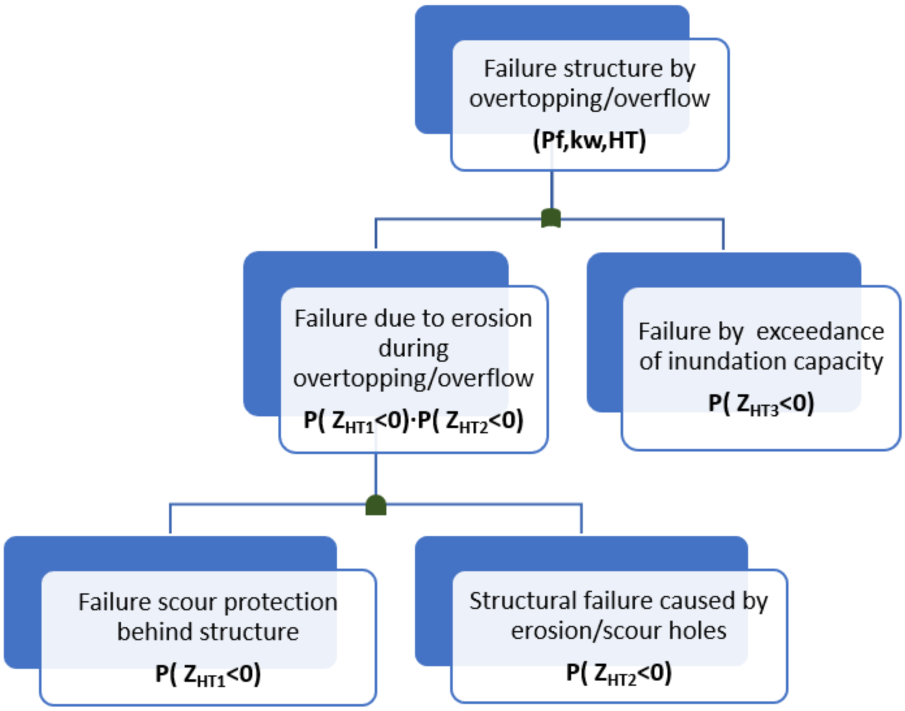


Figure 4.39: Fault tree for the failure process related to wave overtopping and/or overflow

The symbols used in the fault tree in Figure 4.39 are explained below, which relate to the failure events:

$P\ \{Z_{HT1}\ <\ 0\}$	=	the inflow velocity exceeds critical flow velocity of the soil or the scour protection leading to failure
$P\ \{Z_{HT2}\ <\ 0\}$	=	instability of structure caused by scour holes leading to failure
$P\ \{Z_{HT3}\ <\ 0\}$	=	exceedance of inundation capacity in hinterland by inflow
$P_{f,kw,HT}$	=	probability of failure of structure with respect to height by overtopping and/or overflow

, in which the denotations have the following meanings:

Z	=	limit-state
HT	=	failure mechanism: height
f (in $P_{f,kw,HT}$)	=	failure
kw (in $P_{f,kw,HT}$)	=	structure (in Dutch: 'kunstwerk', abbreviated to 'kw')

The fault tree in Figure 4.39 indicates that there is failure caused by insufficient retaining height if the amount of water that inundates the area, caused by overtopping or overflow, exceeds the storage capacity leading to a flood event or if the structure fails because the soil surrounding the structure has been eroded by flow due to wave overtopping or overflow. However, the most governing failure event is the erosion of the soil, since no scour protection is applied and the critical flow velocity for the soil is low, namely u_c = 0.1 m/s (Rijkswaterstaat, 2021).

In this verification, it is assumed that when erosion of the soil or bottom protection occurs, the structure itself instantly fails because of instability, meaning that $P(Z_{HT2} < 0) = 1.0$. The reason is that erosion is not generally preferred and, moreover, this assumption leads to a conservative approach.

This means that the following holds:

$$P_{f,kw,HT} = P\{min(Z_{HT1<0}; Z_{HT3<0})\} \tag{4.1}$$

Also, this failure probability needs to be within limits of the failure probability requirement for the height, determined in the basis of design, which was:

$$P_{f,kw,HT} < P_{req,kw,HT} = 0.0024 \tag{4.2}$$

From this requirement, the fact that erosion of the soil is the most governing and the probability that overtopping leads to a flow with a velocity smaller than the critical velocity of the soil of u_c = 0.1 m/s (Rijkswaterstaat, 2021), the hydraulic load level is calculated with the help of the software Hydra-NL published by Rijkswaterstaat (2021). This hydraulic load level indicates the minimum height of the water retaining structure in order to have a failure probability that complies to the requirement regarding this failure mechanism.

The result of this is a hydraulic load level of NAP + 18.23 m for a functional design lifetime of 40 years with climate scenario W+ predicted by the KNMI (2015). The ground surface elevation at the location of the self-closing flood barrier will be levelled to NAP + 15.4 m, which means that the flat gate should have a height of 2.83 m. The corresponding local design still water level to this hydraulic load level is NAP + 17.84 m with a significant wave height of 0.25 m.

The complete elaboration and reasoning of this verification can be found in Appendix J. In Figure 4.42a a schematisation is shown of the final result of the verification including the definitive height of the self-closing flood barrier along with the boundary conditions.

4.5.2. Determining the dimensions of the floater

This section will elaborate on an estimate of the height and the width of the floater of the barrier. The height and the width of the floater depends mainly on the required buoyancy in order for the barrier to be afloat. As is already concluded in Section 4.4.2, the (partially) immersed volume per unit meter span or cross-sectional area of the floater should be at least 7.85 larger than the cross-sectional area of the barrier, to induce sufficient buoyancy. Firstly a required estimate for the cross-sectional area of the barrier of 0.1 m² was assumed. The final volume of the barrier per unit meter span results in 0.145 m³/m which eventually follows from the structural design in Chapter 5. This means that minimally a volume of displaced water per unit meter span of 1.14 m³/m is required. From this, the height and width is chosen. Import aspects herein are:

- The width should be larger than the height:
  - it allows for more static floating stability, see Appendix L.2;
  - the overall structure requires already sufficient width for vertical, rotational and horizontal stability resulting in the obvious choice to utilise the available width inside the concrete structure for the floater rather than increasing the height
- The overall area of the floater should be larger than the minimally required area of the displaced water (floater should be able to induce more buoyancy than calculated):
  - the barrier is subjected to friction resulting from the rollers and the seals of adjacent barriers;
  - the barrier should be afloat before the water level in the concrete chamber reaches the top of the floater, because water on top of the floater in the initial stage of closing may result in inability to move upward, but also tilting and jamming;
  - the barrier is adaptible and takes into account the potential need for an increase of the retaining height, resulting in heightening of the retaining wall with additional steel leading to more weight
- The height of the floater should be limited as much as possible to limit the overall height of the structure because the retaining height of the barrier as determined in Section 4.5.1 is already 2.83 m to which is added the additional reserved height for adaptivity and the concrete floor thickness. Nevertheless, it does not mean that the width of the floater can be infinitely long. It is naturally an integral approach.

To come to a suitable combination of height and width, an initial method in Appendix L.1 is used to calculate a first estimation of the width assuming the thickness of the floater, the height of the floater that is immersed in water and the cross-sectional area of the retaining wall. This resulted in a width of $2.8\ m$ which is assumed to be reasonable. Thus keeping this value fixed and with the minimally required volume of displaced water, the minimally required height of the floater then becomes 0.41 m. Taking into account the aforementioned attention points, yields the final height of 0.66 m, which means an additional height of 0.25 m that is taken into account as to compensate for the listed attention points. In Figure 4.40 a draft is shown in which the floatibility is illustrated.

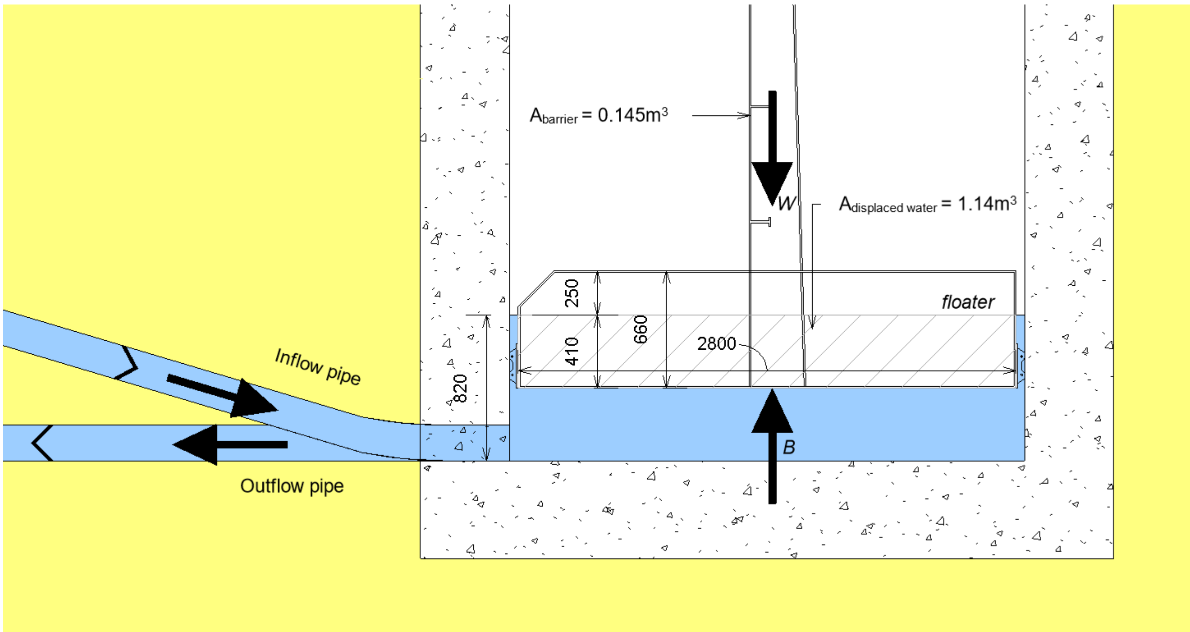


Figure 4.40: Schematisation of floating barrier with dimensions

4.5.3. Summary of preliminary dimensions

In Figure 4.41 the dimensions of the structure are summarised. Note that the total width is 3.9 m and the total height is 4.75 m including all contributors. The thickness of the concrete is justified in the Structural Design (Chapter 5).

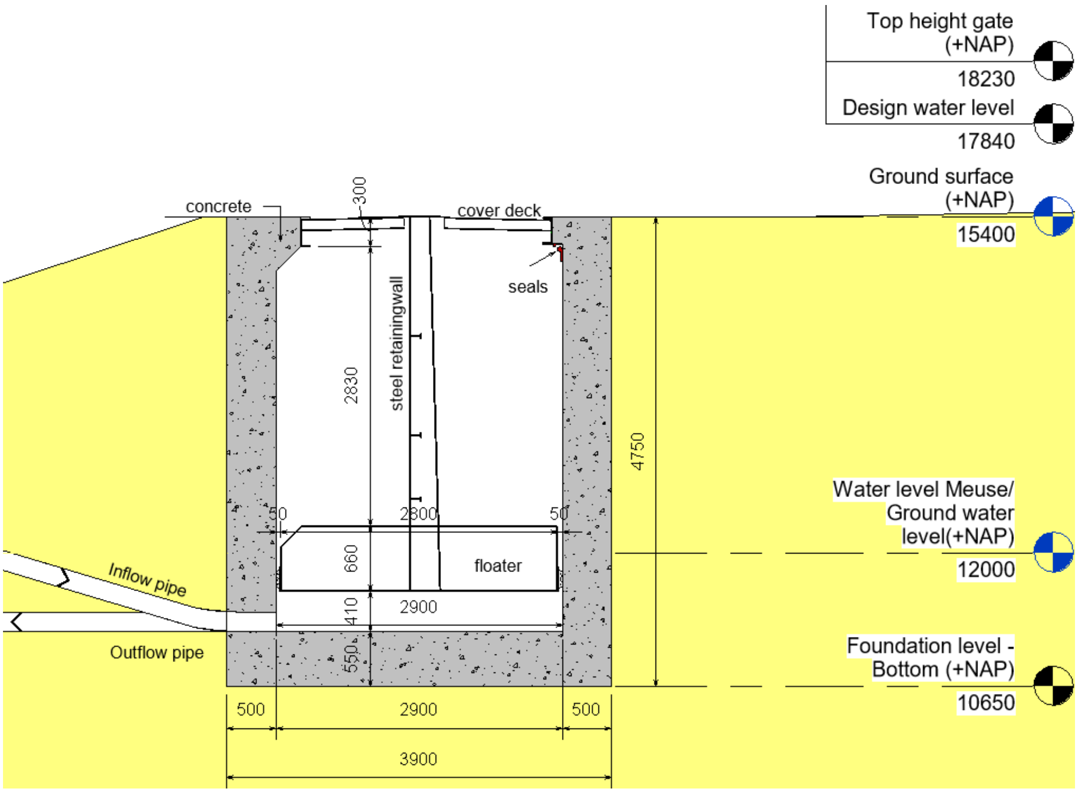


Figure 4.41: Schematisation of final result of preliminary dimensions

4.5.4. Dimensions from a top view

In Figure 4.42 a top view of the barrier location is shown. In Figure 4.42a the location of the barrier is shown in red. In Figure 4.42b a top view of the barrier is shown in green. In blue the current flood defence wall is indicated. Additionally, the distances along the span between this current structure and the self-closing flood barrier are shown. This illustrates also the distance between the barrier and the private properties of residents in the area, as the current retaining wall is very much in close proximity to the houses. The span of the entire barrier for the dike section is 240 m and is subdivided in six closing gate parts of each 40 m span. The reasoning for this is further explained in the next section with the determination of the failure probabilites for the closing process.

(a) Overview of project plan with topography map

(b) Overview of plan with measurements

Figure 4.42: Overview of project plan

4.6. Determining probabilities of components regarding non-closure

Besides the global dimensions and the shape of the structure that are determined in the previous sections, it is also important for the functionality of the barrier to determine the maximum allowed failure probabilities of the components with respect to the closing process. The goal of this section is to gain insight in the failure probabilities to which components should be designed for and thus to gain insight in the reliability of the structure with respect to the closure of the structure. The determination is based on a fault tree analysis.

4.6.1. Disclaimer:

In this section failure probabilities will be assigned for components related to the closure of the self-closing flood barrier to assign a certain reliability of the structure with respect to closure in order to perform this verification. It should be pointed out that these numbers are indicatively provided to gain insight in the order of magnitude of the maximum allowed failure probabilities and the critical parts in the design with respect to reliable closure. The numbers are more exactly specifiable when designing the closure components in detail. More importantly in this section is the set-up of the fault tree, the failure events and their relation with respect to each other.

4.6.2. Introduction in the failure mechanism ’non-closure’

The self-closing flood barrier requires closing during an expected high water event. However, the closing process has a certain probability of failure, resulting in not closing, which leads in turn into a potential flood event if also the occurring water level exceeds a certain threshold. This failure probability depends on the selected gate type, the drive mechanism and the functional components of the latter two. Therefore in this section, the closing process of the self-closing flood barrier is designed in terms of failure probabilities to which must be complied to.

Firstly, the failure probability requirement for non-closure is stated and subdivided for one single closing gate. Then, the fault tree and the failure events of the closing process for one single gate are identified. This yields a failure probability requirement for the actual closing mechanism of the gate, but it depends on the closure demands per year, which must be determined first. Then by specifying the fault tree for the actual closing mechanism, insight is given in the failure probabilities to which the essential components and subprocesses should comply to, in such a way that the reliability of the closing process complies to the failure probability requirement regarding this failure mechanism for the closure demands per year.

Failure probability requirement from the standard for non-closure per dike section

The failure probability requirement for the failure mechanism ’non-closure’ derived from the standard in the Dutch Water Act is: $P_{req,HS,NC} = 6.7 \cdot 10^{-5}$ as was mentioned in the Basis of Design in Chapter 3. The length-effect factor herein is $N = 6.0$, because there are five dike sections within the dike segment with each a newly constructed self-closing flood barrier and furthermore, in the current dike segment there is also already a pumping station present, which does not fall within the project area, but takes part in the entire dike segment and is therefore assumed to remain unaffected. This makes the total number of hydraulic structures within the dike segment for the length-effect factor to $N = 6.0$.

Note: $P_{req,HS,NC} = P_{requirement,\ Hydraulic\ Structure,\ Non-Closure}$ in which P is the symbol for failure probability.

Dividing the barrier into individually closing structures within dike section

Within the dike section and thus the self-closing flood barrier of 240 m, the number of closing gate parts is set to $n_{gates} = 6.0$. This leads to a span per gate of approximately $40m$. This is a reasonable span length for gates with this hydraulic head, compared to reference projects (Daniel & Paulus, 2019). Decreasing the gate span length, i.e. increasing the number of gates leads to a more favourable condition for the structural design but more unfavourable for the reliability of closure, because there are more independently closing parts. Increasing the gate span length, i.e. decreasing the number of gates leads to a more unfavourable condition for the structural design but more favourable for the reliability of closure, because there are less independently closing parts. However, failure in such a case leads to larger consequences for the area. With $n_{gates} = 6.0$, i.e. a span length of 40 m both conditions seem to be balanced.

In conclusion the requirement to be verified for reliable closing of a self-closing flood barrier is $P_{req,HS,NC} = 6.7 \cdot 10^{-5}$, which is to be divided into $n_{gates} = 6$ closing parts.

Fault tree for structure not closing

The fault tree for the structure not closing is given in Figure 4.43. With the use of the fault tree the failure probability of the closing mechanism, $P_{f,CM}$, will systematically be derived.

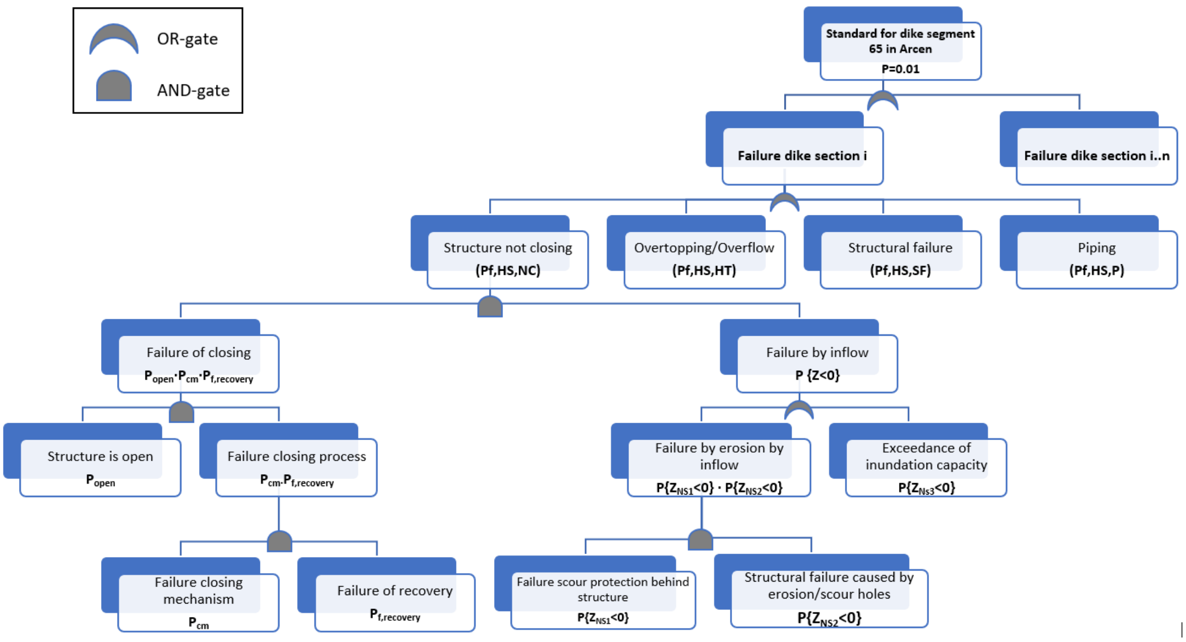


Figure 4.43: Fault tree for the failure process related to reliable closure

There are five main events that determine the failure probability for not closing of the structure:

- The structure is open, with an expected high water event, which means that there is a closure demand. (P_{open})
- Failure of the closing mechanism which relates to the process from alarming to the technical closing ($P_{f,CM}$) and the failure of its corresponding recovery ($P_{f,recovery}$)
- Failure of the scour protection behind the structure because of inundation ($P\ \{Z_{NC1} < 0\}$)
- Structural failure caused by scour holes and erosion due to the scouring process ($P\ \{Z_{NC2} < 0\}$)
- Exceedance of the inundation capacity in the area behind the structure ($P\ \{Z_{NC3} < 0\}$)

Combining all events contributing to the failure probability of not closing of the self-closing flood barrier results in the following product of the contributing probability factors:

$$P_{f,HS,NC} = P_{f,CM} \cdot P_{open} \cdot P_{f,recovery} \cdot P\ \{Z < 0\} \tag{4.3}$$

, where the failure due to inflow, ($P\ \{Z < 0\}$), depends on either exceedance of inundation capacity (Z_{NC3}) or failure by erosion (Z_{NC1}). Herein, it is assumed that structural failure caused by erosion always occurs, thus $P\ \{Z_{NC2}\} = 1.0$.

In equation 4.3, it is further assumed that the structure is always open ($P_{open} = 1.0$) and recovery measures always fail ($P_{f,recovery} = 1.0$), in order to take a conservative approach. The probability of failure of the closing mechanism ($P_{f,CM}$) and the probability of inflow ($P\ \{Z < 0\}$), which in fact is the number of closing demands per year, remain to be determined. These are elaborated below.

Determination of the required probability of inflow ($P\ \{Z < 0\}$; closure demands per year)
The probability of failure of not closing depends on the number of closure demands per year, because the structure should only close in case of a high water event, which has a certain probability of occurrence of its own. Failure of closing the water barrier in combination with a water level in the Meuse that does not have a consequence of flooding the hinterland or erosion of the soil near the structure is not labelled as failure by not closing (Rijkswaterstaat WVL, 2021). Being labelled as failure depends on the probability of a certain outer water level that either leads to:

- exceedance of the water storage capacity in the hinterland by an inflow through the opening(s) (Z_{NC3}) or;
- an inflow with a flow velocity exceeding the critical flow velocity of the soil or bottom protection causing erosion of the soil near the structure (Z_{NC1}).

The governing situation herein is erosion, because scour protection is not present in the area and is also not preferred, because of the well known fact that the barrier is located in private gardens and maintaining the spatial quality is of importance. Thus, the closure demands per year is equal to the probability of occurrence of a maximum allowed outer water level that induces flow through the barrier opening with a flow velocity exceeding the critical flow velocity of the soil. However, since no scour protection is present, any type of flow on the soil is not allowed, because fine sand has a very low critical flow velocity, i.e. highly erodible (Rijkswaterstaat WVL, 2021). This results in that the maximum allowed outer water level corresponding to a closure demand will be at NAP + 15.4m, which is equal to the ground surface level at the location of the self-closing flood barrier. The probability of occurrence of this outer water level is obtained by a frequency line from Hydra-NL showing the return periods for water levels in the particular region of the Meuse. The probability of occurrence associated with this water level that is determined is $P\ \{Z < 0\} = 0.22$. The full elaboration of the determination of required probability of inflow is given in Appendix K.

The probability of inflow and thus the number of closure demands per year is $P\ \{Z < 0\} = 0.22$. This means that per 4.5 year the self-closing flood barrier needs to be closed once on the average. However, the probability of occurrence is determined with current statistics and prediction models, which can have a different outcome in five years. For example, due to climate change, future water levels in the Meuse could be higher, leading to a higher number of closure demands. This would mean a higher probability of non-closure per year, which in turn would result in stricter requirements for the design of the closing mechanism.

Now that the closure demands per year is determined, the last contributor in Equation 4.3 can be determined, which is the failure probability of the closing mechanism $P_{f,CM}$. This is elaborated in the following paragraph.

Determination of the required failure probability of the closing mechanism
This section shows the determination of the required failure probability of the closing mechanism resulting from the other contributing factors taken into account as determined previously to the failure probability requirement for the failure mechanism of not closing. The requirement derived from the standard of the Dutch Water Act is determined in the basis of design in Chapter 3 for $n_{gates} = 6.0$ and should not be exceeded, meaning the following equation must hold:

$$P_{f,HS,NC} \leq P_{req,HS,NC} = 6.7 \cdot 10^{-5} \tag{4.4}$$

The probability of occurrence of not closing is determined with Equation 4.3, from which only $P_{f,CM}$ is the unknown:

$$P_{f,HS,NC} = P_{f,CM} \cdot P_{open} \cdot P_{f,recovery} \cdot P\ \{Z < 0\} = P_{f,CM} \cdot 1.0 \cdot 1.0 \cdot 0.22 = 6.7 \cdot 10^{-5} \quad (4.5)$$

which means that the verification reliable closing is in agreement, when the required probability of failure of the closing mechanism is:

$$P_{f,CM} = \frac{P_{f,HS,NC}}{P_{open} \cdot P_{f,recovery} \cdot P\ \{Z < 0\}} = \frac{6.7 \cdot 10^{-5}}{1.0 \cdot 1.0 \cdot 0.22} = 3.03 \cdot 10^{-4} \quad (4.6)$$

Design of barrier with accepted probability of failure of closing mechanism ($P_{f,CM}$)
The previous paragraph yielded a requirement for the failure probability of the closing mechanism, which is $P_{f,CM} = 3.03 \cdot 10^{-4}$. This holds for the entire barrier of 240 m. However, the barrier consists of individually closing gate parts which are identical, meaning $n_{gates} = 6.0$. The requirement is thus equally divided by $n_{gates} = 6.0$ for the failure probability of the closing mechanism per gate which yields $P_{f,CM,i} = 5.05 \cdot 10^{-5}$. Per individual closing gate part the probability of failure of the closing mechanism can be translated to a fault tree showing the contributing failure processes. There are two methods for achieving this, namely a standardised method with a generic fault tree and an advanced method with a customised fault tree specifically for this design of a self-closing flood barrier. Each method is elaborated for comparison and briefly described below with the outcomes regarding the design with respect to the closing mechanism:

Standardised method:
The standardised method is a conservative method that uses a scoring table for generic closable hydraulic structures in the Netherlands which does not take into account customised changes to a structure (Casteleijn & van Bree, 2017). For this reason, the standardised method has a lower limit for the probability of failure regarding the closing mechanism. With the standardised method the following probability of failure of the closing mechanism per gate part is minimally achievable: $P_{f,CM,i} = 3.2 \cdot 10^{-4}$. Thus, this method cannot verify that the design meets the failure probability requirement for the closing mechanism, when it might does actually comply. The complete elaboration of the standardised method with the scoring table and fault tree is given in Appendix K.

Advanced method:
The advanced method uses a customised fault tree for the closing mechanism that takes into account design choices for the structure directly affecting the probability of failure. With the advanced method the accepted failure probability for the closing mechanism per individually closing gate is achieved by adjusting the design, so that $P_{f,CM,i} = 5.05 \cdot 10^{-5}$. This failure probability takes into account a certain closure protocol.

The associated fault tree for this method is given in Figure 4.44.

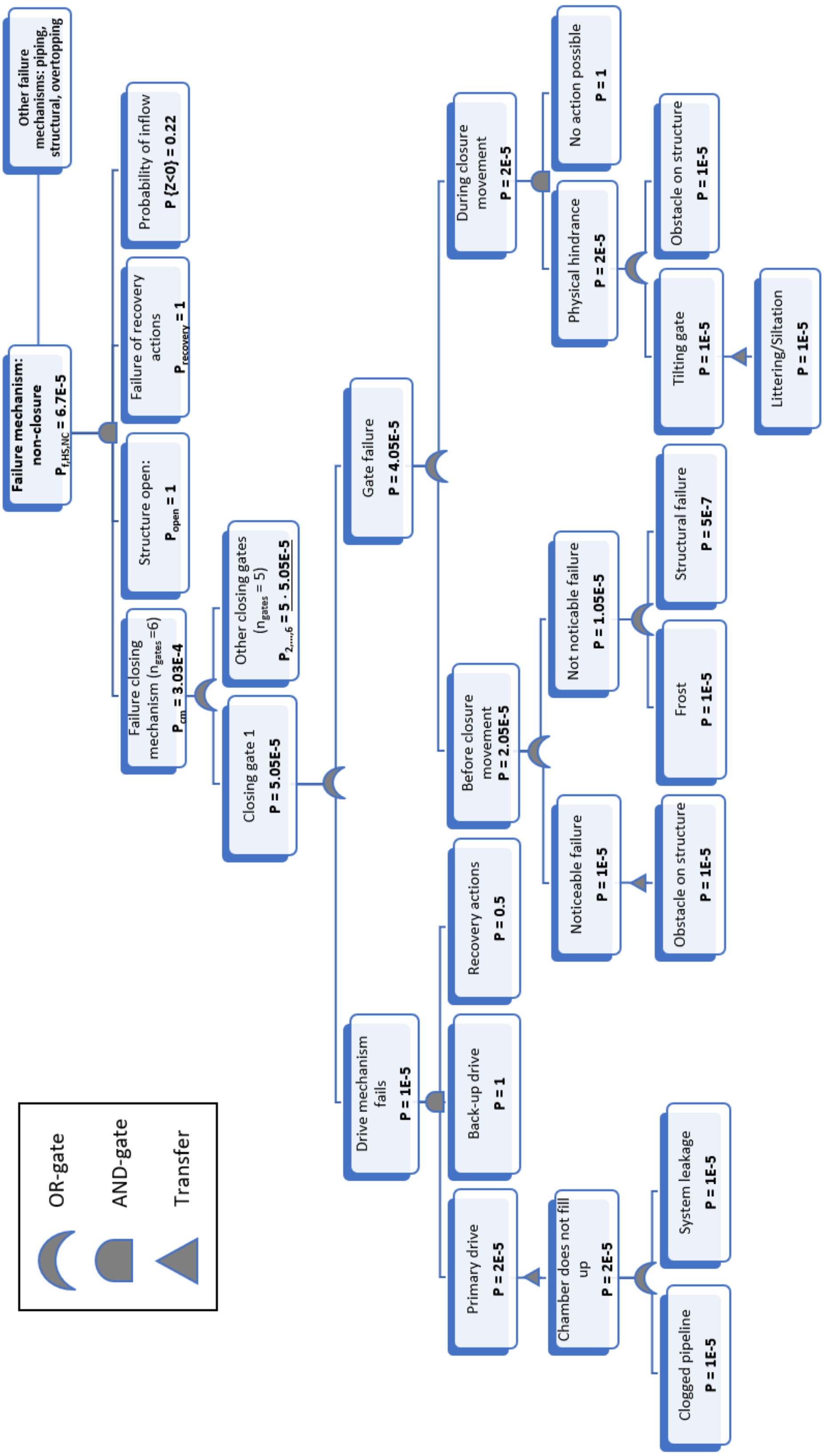


Figure 4.44: Fault tree for the failure of the closing mechanism

Closure protocol
It is taken into account that there is a back-up drive to fill the system in case the primary drive is not available and that the gate is manually closable in case of an emergency recovery action. Furthermore as part of the inspection, test and maintenance regimen, it is assumed that during an anticipated high water event, as part of an inspection regime applicable to the control of this barrier, an inspection is performed two hours prior to a closure demand to remove obstacles such as large flower pots or other physical obstructions, if any are present. Furthermore, twice a year maintenance is performed to the structure to remove debris in the filling system and to check the condition of structural components. Lastly, the closing mechanism is once a year tested as well.

Conclusion on design with failure probability of closing mechanism
As was mentioned in the disclaimer in the beginning of this section, the assigned failure probabilities provide an order of magnitude which are considered reliable based on a comparison with failure probabilities of external events from the ANSI/ANS 58.21-2007 norm method (ANSI = American National Standard Institute) used by Rijkswaterstaat (van Bree & Casteleijn, 2017). This means that a design for a self-closing flood barrier regarding the reliability of the closure mechanism as such is considered feasible. Additionally, in the fault tree of Figure 4.44 can be seen that the failure probability of the back-up drive and the recovery action are assumed to be very high, which in reality would not be the case. This is merely done to take a conservative approach and to show that the failure probability of the primary drive mechanism has a certain margin in the design.

Furthermore, from the resulting fault tree in Figure 4.44, it can be concluded that with a design for a self-closing flood barrier as such, the failure processes should have an individual failure probability with an order of magnitude as indicated in the fault tree in Figure 4.44. This can be achieved by selecting components available on the market that contribute to this by having features that reduce the probability of occurrence of certain failure processes, such as a pipe with filter to prevent clogging or adding a heat element to prevent frost. Alternatively, components associated to the failure processes should be tested and designed in such a way that they comply to the failure probability requirements. In conclusion, for a potential follow-up design, a detailed fault tree analysis should be performed with scientifically or statistically supported values for the failure probabilities. In this thesis no further elaboration is done on this.

Using the standardised method, the failure probability requirement for the closing mechanism is not achievable. In fact, the minimal resulting failure probability is six times larger than the requirement, in contrast to the advanced method. The standardised method is generic for all hydraulic closing structures and is more conservative, meaning that the difference between the two methods is not surprising. Comparing the two methods, results in the decision to use the advanced method because it represents the actual situation more than the standardised method does. One of the reasons is that a self-closing flood barrier is a very specific closable hydraulic structure that needs to take into account design features for the reliability of closure, that are not taken into account in the standardised method. More elaboration on this comparison and the accuracy of the advanced method can be found in Appendix K.

4.7. Making the functional design adaptive

An important focus point in this design study is the feature to adapt the design to new hydraulic boundary conditions. This influences the height of the overall structure, which is already shown before in Figure 4.41. In this section, the contribution of the adaptibility to the height of the structure will be elaborated on, by firstly discussing the influence on the design, then the guiding principle used for determining the boundary conditions and lastly the actual execution of adapting the design.

4.7.1. Influence of adaptibility on the design of the self-closing flood barrier

In this project, the adaptibility will be considered for the retaining height of the structure. The functional lifetime as stated in the Basis of Design in Chapter 3 is 40 years and after 40 years of lifetime the structure is re-evaluated to assess if the structure satisfies the new hydraulic boundary conditions for the following functional lifetime. The assessment may lead to the need for adapting the design or to no adjustments to the structure at all, which would be the most desirable outcome.

4.7.2. The guiding principle for the boundary conditions for the adaptive design

The Guideline Design Hydraulic Structures states that a hydraulic structure should be designed with climate scenario W+ for the end of the functional lifetime (van Bree et al., 2018). However, for an adaptive design it is allowed to adhere to climate scenario G or G+ (van Bree et al., 2018). This is on the other hand not the case for the foundation. In this design study for a self-closing flood barrier, W+ will be the climate scenario for which the adaptive structure will be made for a functional lifetime of 40 years. This functional lifetime coincides with the same hydraulic boundary conditions as if there were to be designed for a lifetime of 100 years with climate scenario G. Conveniently, this is in accordance with the guideline, since it is stated that for an adaptive design, climate scenario G can be used for the end of the functional lifetime. Note that this does not apply to the foundation. For the foundation, the end of the lifetime is 100 years, which is typical for structures in general.

4.7.3. The boundary conditions for an adaptive design

In order to determine the adaptibility in the retaining height of the structure, the required retaining height for a functional lifetime of 100 years with climate scenario W+ is evaluated. This is the most extreme situation that Hydra-NL is able to predict with the current statistical data. So assuming that in 100 years extreme climate changes would occur, a retaining height for the project location of 3.24 m would be necessary assuming that the elevation in the area is equal to the current situation. So this would be the retaining height for a design with lifetime of 100 years without adaptibility taken into account. This is considered as the upper limit value for the retaining height of the structure. However, the design for the self-closing flood barrier is made adaptive but with a functional lifetime of 40 years with climate scenario W+. This yields a retaining height of 2.83m. Comparing the retaining height for the design in this thesis and the upper limit value, the required retaining height adjustment after a lifetime of 40 years could possibly go up to 3.24 m for a second lifetime, depending on future statistics and predictions. In conclusion, the design of the self-closing flood barrier will be made adaptible to a retaining height of 3.24 m. The concrete chamber should therefore have a minimum height of 3.24 m on the inside. For the total inside height in the concrete chamber, the height of the floater needs to be added still. This was already shown in Figure 4.41.

4.7.4. Adapting the structure of the self-closing flood barrier

A possible adapted design involves welding an additional plate element on top of the barrier leading to simply an increased retaining height. This adjustment will also result in more weight which requires more buoyancy. So the floater design needs to take into account an additional weight. This is already compensated for in dimensioning of the floater in Section 4.5.2.

4.8. Integrating the barrier in the surroundings

In Figure 4.45 the area is shown of dike section 2 as indicated before in Figures 4.1 and 4.42. The structure is lowered in the concrete chamber and integrates nicely with the ground surface. The covers make sure that the gate recess is closed off and can be walked on. The red wall is the current flood defence structure that crosses the gardens. They can be removed since the barrier is now the flood defence. The grey block in the right lower corner represents the Meuse Terrace. This is actually the end of the flood defence section. In Figure 4.46 a better representation is given, where a concrete structure is constructed next to the wall of the Meuse Terrace with a recess for the gate end with seal to connect. The red buildings in the back are houses.

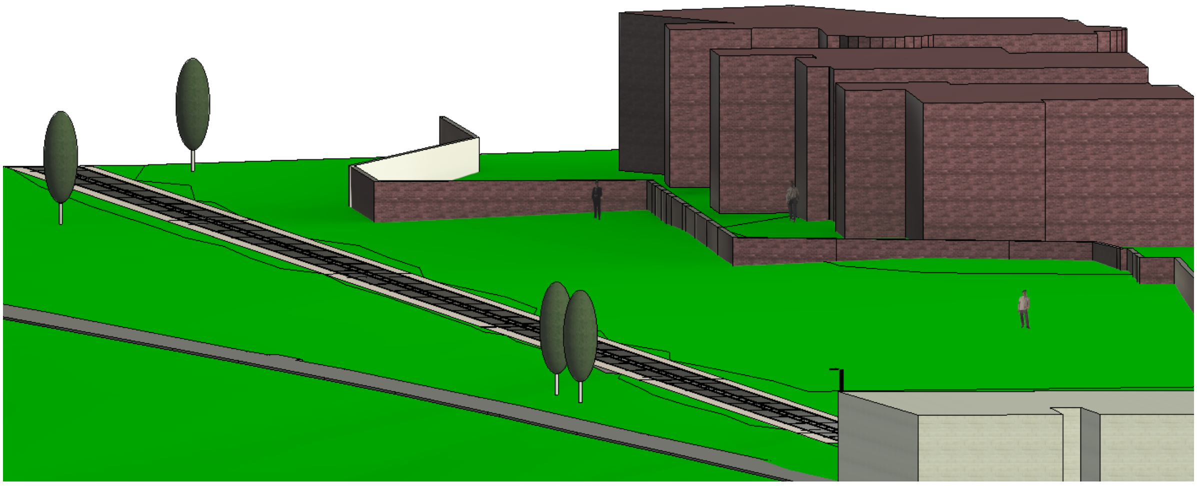


Figure 4.45: 3D impression of the project area and the the self-closing flood barrier lowered

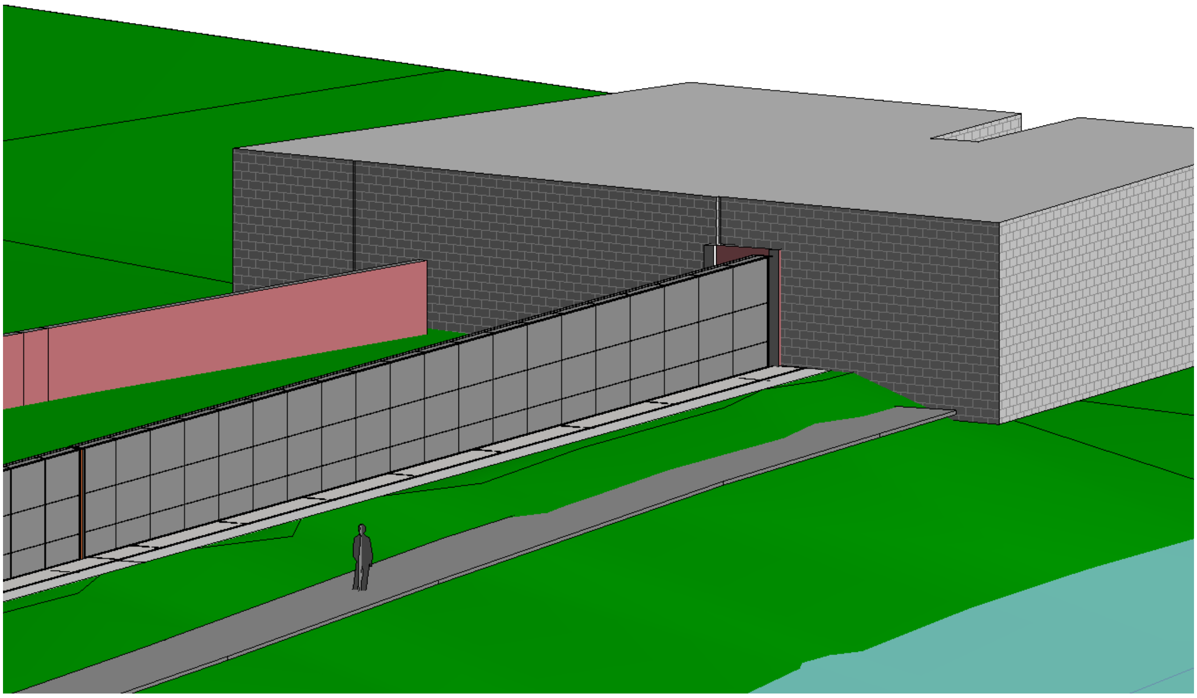


Figure 4.46: 3D impression the end of the flood defence section at the Meuse Terrace

In Figure 4.47 a high water event is shown in which the barrier is fully closed and retains the extreme water level. In Figure 4.48 another view of this is shown.

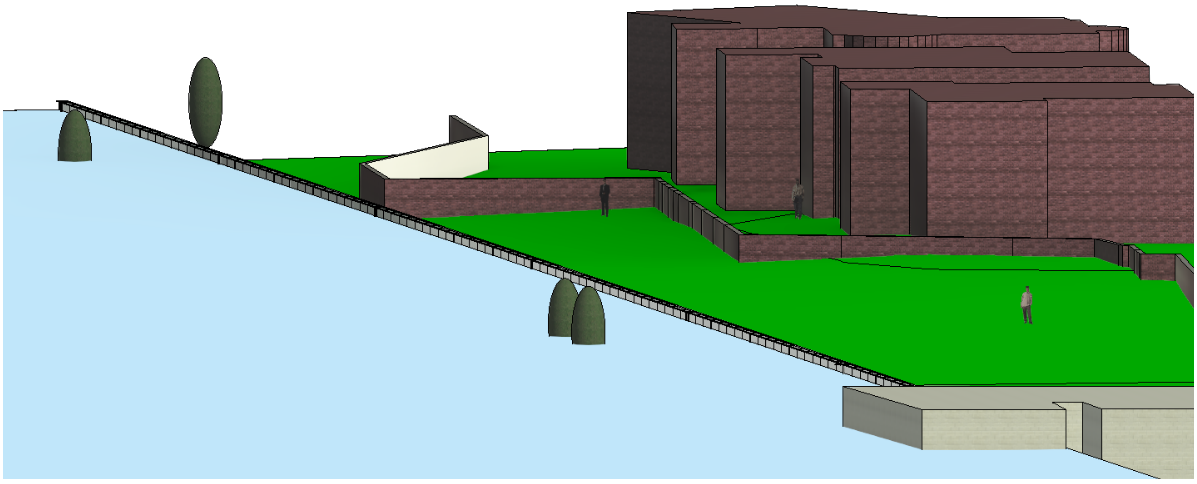


Figure 4.47: 3D impression of the closed self-closing flood barrier during a high water event

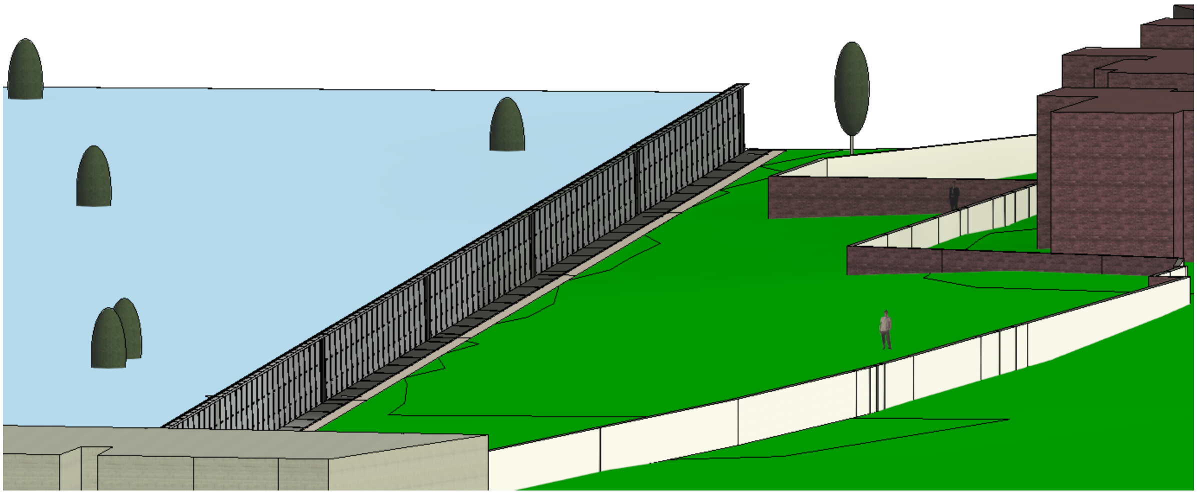


Figure 4.48: 3D impression of the closed self-closing flood barrier during a high water event

In Figure 4.49 a cross-sectional 3D view is shown, where one closing gate is lowered in the concrete chamber and where one closing gate is in its fully closed state. In orange, the connecting seal between two gates can be well seen. Note that the water is not shown inside the concrete chamber in this figure.

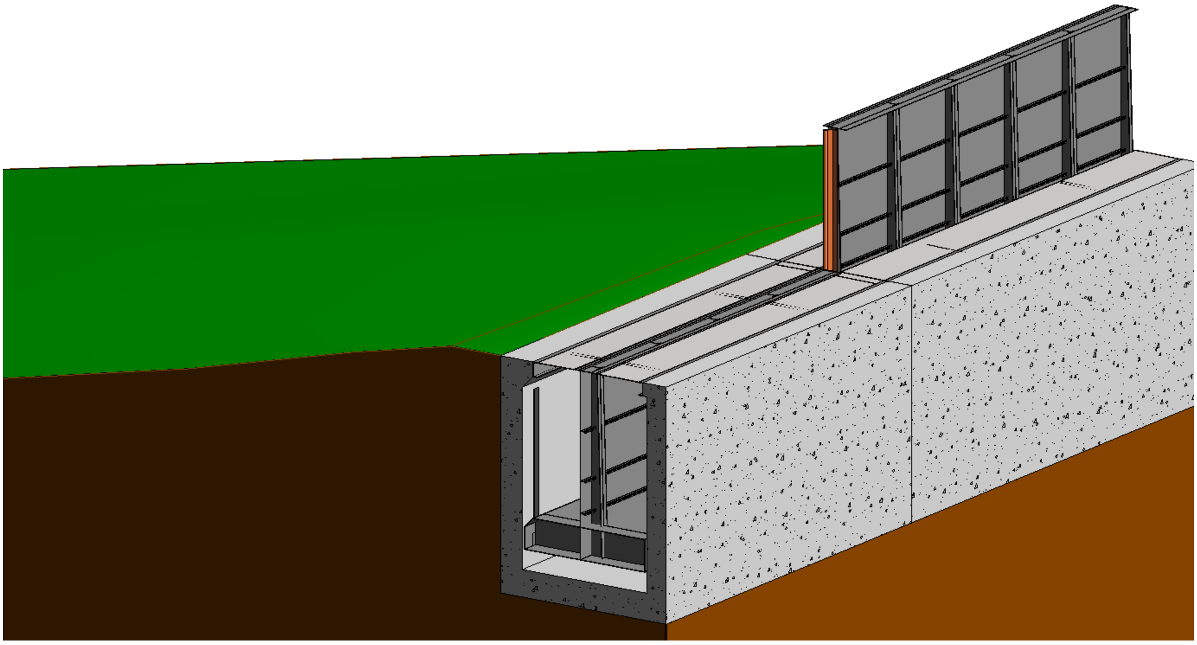


Figure 4.49: 3D impression of the self-closing flood barrier in the project area

In Figure 4.50 a 3D is shown from above in which the covering of the structure can be well seen for one gate. The other gate is shown without cover to give an impression of the set up of the structure.

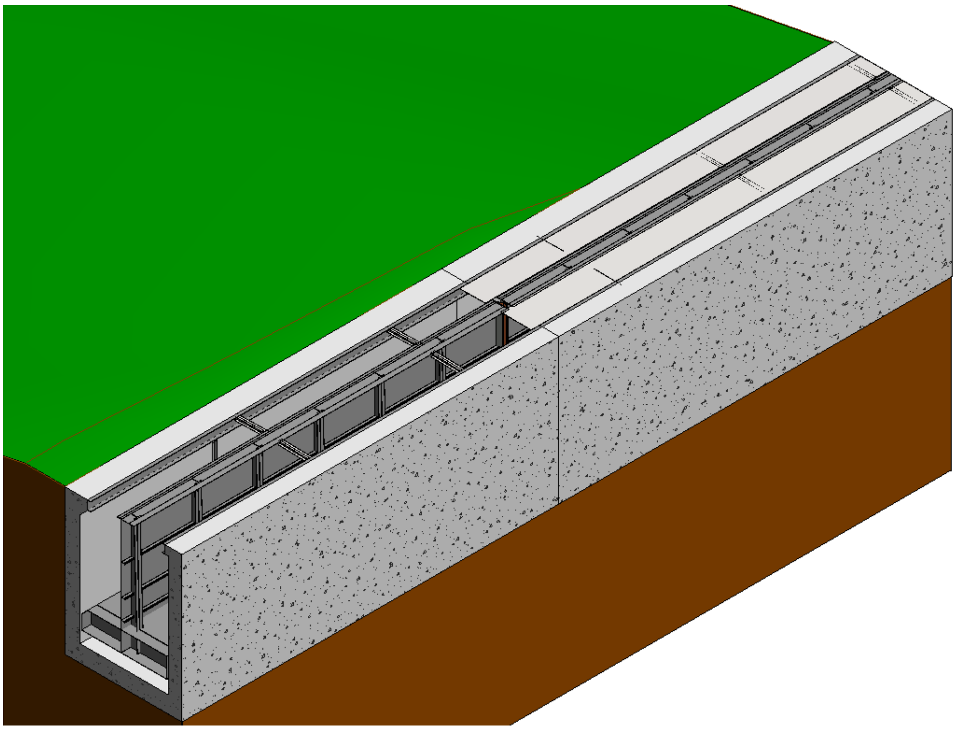


Figure 4.50: 3D impression of the self-closing flood barrier in the project area

5 Structural Design

5.1. Constructibility

5.1.1. Construction method

The construction method is a combination of prefabrication and in-situ. The concrete is casted in-situ, the main structural frame of the steel gate is prefabricated, transported to the construction location and connected to the system on site. In appendix M.1 the justification can be found for the selection of the construction methods.

5.1.2. Excavation technique

Two excavation techniques suitable for the construction of the self-closing flood barrier are considered, namely a construction pit with natural slopes and a cofferdam bounded by sheet pile walls. In Figures 5.1 and 5.2 the two excavation techniques are illustrated. A construction pit with natural slopes takes relatively much space which is not suitable in an urban area as such (Molenaar & Voorendt, 2023), whereas a cofferdam take less space and is thus the more forward option regarding this aspect. In conclusion a cofferdam is chosen as excavation method, even though it is a more expensive alternative. A full justification for the selection of the excavation technique is given in Appendix M.2. A cofferdam can be executed with an underwater concrete floor or with the help of drainage (Molenaar & Voorendt, 2023). In this project is chosen for an underwater concrete floor, since the floor needs to be casted anyways and tension piles may not be required since the upward ground water pressure is not high.

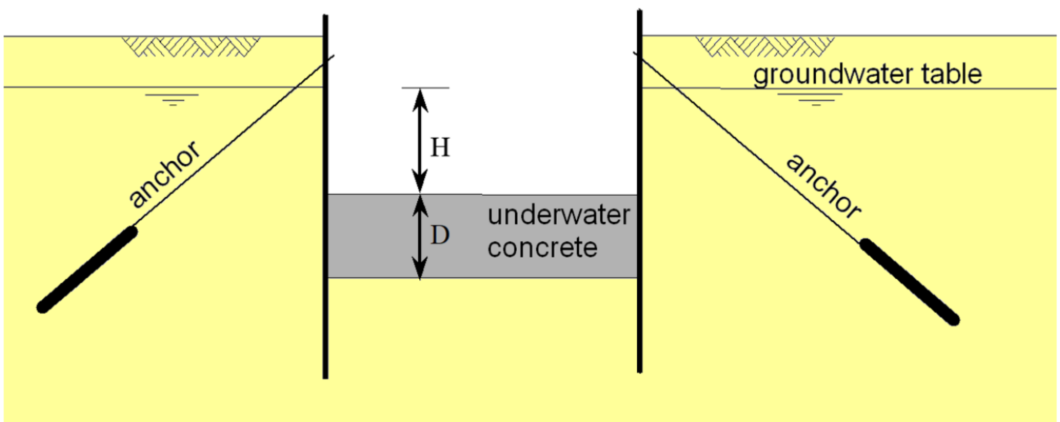


Figure 5.1: Example of schematic cross-section of cofferdam (Molenaar & Voorendt, 2023)

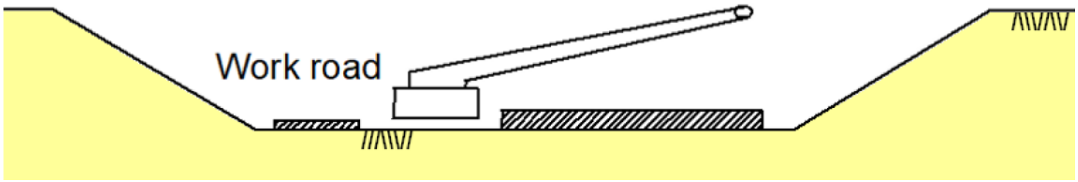


Figure 5.2: Example of schematic cross-section of construction pit (Molenaar & Voorendt, 2023)

5.1.3. Foundation method

For the foundation method there are two main possibilities, namely a shallow foundation or a pile foundation. The structure is embedded into the soil with a depth of approximately 4.75 m, so excavation is already required. It is important to check what the soil conditions are at the foundation depth and below to decide which foundation method to choose. In the Basis of Design in Chapter 3, the boundary conditions related to the soil structure were already covered in which it was pointed out that the soil consists of mainly fine sand (DINOloket, 2022). The CPT graphs for the area, derived from DINOloket (2022), are shown in Appendix N. From these CPT graphs it can be seen that at a depth of 4 m and lower below the ground surface level, the soil has sufficient cone resistance (> 5-8 MPA) and thus consists of already load bearing sand layers on which directly can be founded. Furthermore, settlements are unlikely to occur since the weight of the excavated soil is larger than the substituted weight of the structure. This will be briefly covered in the verification of the vertical stability in Section 5.3.1 which also elaborates on the bearing capacity. For the same reason, uplift in the governing case will not occur since the ground water pressure does not exceed the weight of the structure. Similarly, this is elaborated further in the report at the verification of uplift in section 5.3.4.

For this reason a pile foundation is not necessary. So the structure will be founded with a shallow foundation, which is elaborated further in Section 5.3. The structure is relatively small in width and because a horizontal sealing for the cofferdam is needed, since no impermeable layers are present, a slab foundation is chosen for this on which both bearing walls of the concrete chamber can transfer the loads. This design choice is also in accordance with the excavation technique in which was already mentioned that an underwater concrete floor has the preference. Since the foundation depth is already 4.75 m below ground level, it is safe to assume that the frost line is above the foundation depth because the minimum depth to prevent freezing of the soil is often 0.6 to 0.8 m below ground level.

5.1.4. Transport and logistics

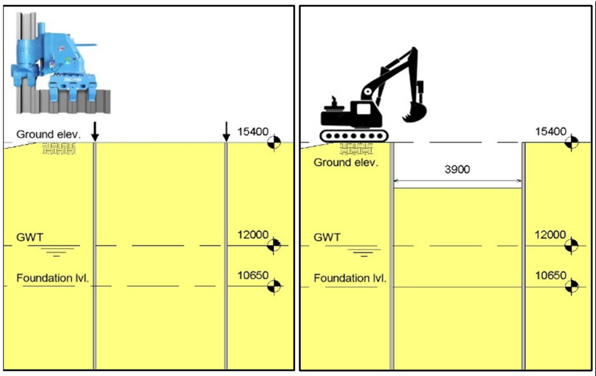
As has already been pointed out in the location selection in Section 4.1, the construction site is located in private backyards along the Meuse over a stretch of more than 200 m. This means that a part of the private gardens must be used for construction works, such as excavation, sheet piling and concrete casting. For this construction hindrance the residents need to be informed and compensated to restore their gardens after the construction works are finished. Further study on this is outside the scope of this thesis.

In view of reducing the construction hindrance and minimising the effect on the spatial quality during the construction period, there are few aspects to consider. Because of limited space in the area, the construction area will be located solely on the river side of the barrier. The touristic promenade along the Meuse will be open from the south up to the 'Maasterras', what is marked as the end of the dike section in question. Transport and short-term storage for construction materials, structural components and equipment can be stored at a small site of 0.1 ha which is located in the northern corner of the Burgemeester-Linders Promenade which is accessible from the Maasstraat. The construction area is good accessible from the north via the part of the Maasstraat which is a rural road connected to the provincial road N271. Transport via these roads will minimise the construction hindrance.

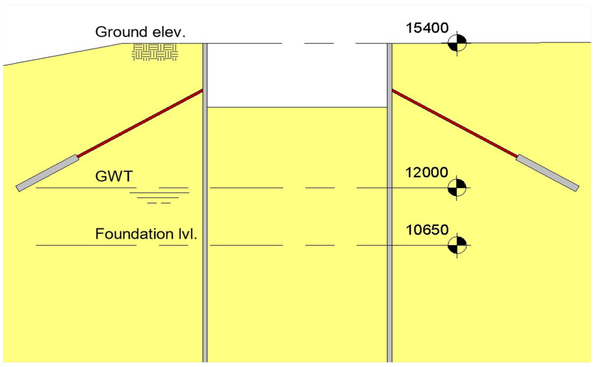
The complete elaboration regarding the transport from and to the construction site is given in Appendix M.4 with associating maps supporting this.

5.1.5. Construction sequence

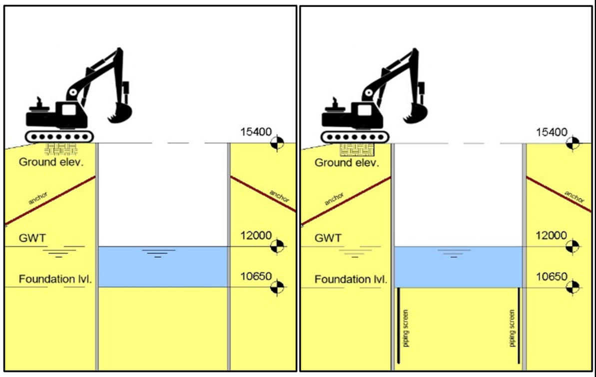
The presented construction sequence only takes into account the construction of the main civil structure involving the process from excavation to casting the concrete chamber and installing the gate. This means that the construction of the pipe network is omitted here. In the following 2D figures the construction steps are summarised with a description. The complete description can be found in Appendix M.5.



1. Sheet piling and initial excavation
Firstly, sheet pile walls are installed inside the soil at the required embedded depth. These sheet pile walls function as the vertical walls of the cofferdam. Since the construction area is in the proximity of existing buildings consisting of mainly houses, it is preferable to minimise the construction hindrance, by using for example silent pilers that press-in instead of vibrate. When the sheet pile walls are in place, the cofferdam can be excavated to a certain intermediate level. The cofferdam will not be fully excavated because the sheet pile wall need to be supported first in order to ensure their stability.

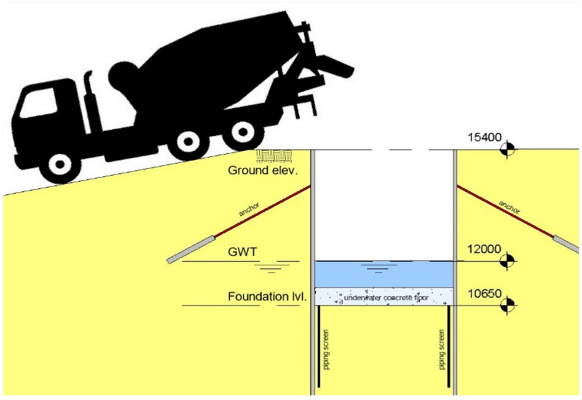


2. Installing grout anchors
The sheet pile walls will be anchored in the soil in order to prevent the walls from being pushed away by the horizontal active soil pressure.

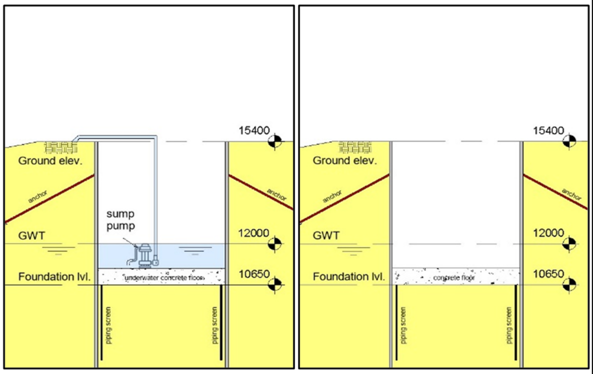


3. Excavation to completion
The cofferdam is now ready to be excavated to its ultimate level. Medium sized excavators are able to excavate to a depth between 5 - 7 m. So excavation can be done from the ground surface. The excavated soil will be used to level the surface around the structure to NAP + 15.4 m. The final excavated level lies below the ground water table, which means that water will flow in to the cofferdam with a water depth of approximately 1.35m. The next step is to place the piping screens into the ground beneath the foundation level where structure will be constructed.

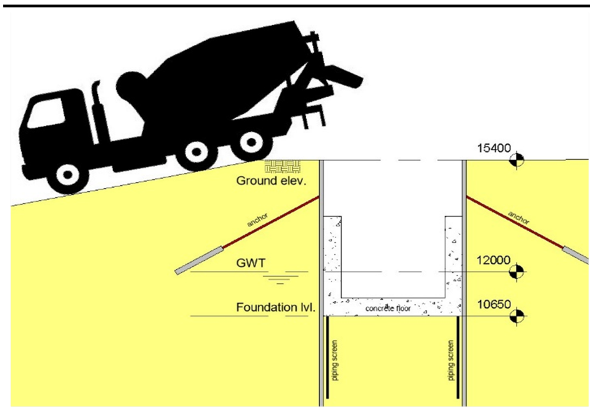
Figure 5.3: Construction sequence



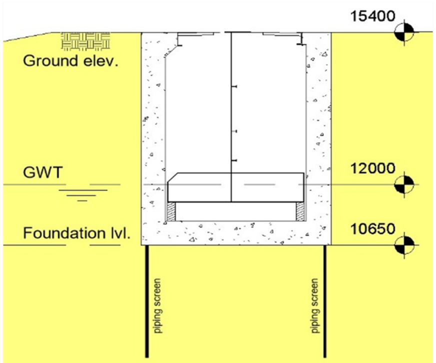
4. Casting of underwater concrete floor
The cofferdam needs to be sealed from the bottom in order to create a dry pit. This is done by casting an underwater concrete floor as a horizontal sealing. Note: tension piles may be needed also, but this will follow from the vertical stability verification. Tentatively, this step is excluded from this construction sequence.



5. Dry pumping cofferdam
When the underwater concrete floor is hardened and the horizontal seal is complete, the cofferdam can be pumped dry by sump pumps.



6. Casting of concrete walls
The next step is to cast the concrete walls to create the floatation chamber of the self-closing flood barrier. The underwater concrete floor may be integrated as the floor of the permanent structure. Before the actual casting, the formwork need to be installed in place and the steel reinforcement need to be placed. After concrete hardening and reaching its full strength, the inside of the concrete will be finished for the guidance rollers to ensure minimum friction to the movement of the gate. Also, to the top part of the walls anchor plates will be attached for connecting steel consoles and cover plates in the next step.



7 - 10 Finalisation
In the finalisation, the concrete walls are casted further until the ground level surface. Then the sheetpile walls can be removed. Lastly the gate parts are suspended into the concrete structure, making sure that the guidance rollers are in the right recesses. First the gates are suspended partially, in order tot attach the steel cons.les with top cover deck and seals to the concrete walls with bolts. Afterwards, the recess can be closed of with a steel deck cover plate and the gate parts can be suspended further into the concrete chamber to maximally submerged position. Step 7 to 10 is further elaborated in Appendix M.

Figure 5.4: Construction sequence (continued)

5.2. Determining loads per critical situation

In this section firstly the failure mechanisms are inventoried for two phases, namely the construction phase and the use phase. However, only the failure mechanisms in the use phase will be considered in this thesis. From the failure mechanisms the governing situations will be specified for the failure mechanisms. Lastly, the loads for the critical situations will be determined.

5.2.1. Failure mechanisms

Before the acting forces are identified, firstly the failure mechanisms need to be defined. Herein, a distinction is made between the construction phase and the use phase. Only the failure mechanisms in the use phase will be considered further in the report

Construction phase

Overturning of sheet pile wall
If a sheet pile wall is not embedded with sufficient depth, the equilibrium between horizontal pressures and forces and between moments could be out of balance leading to overturning of the sheet pile wall. The sheet pile wall would not be sufficiently supported by the soil on the passive side and the anchor force to resist the acting horizontal forces on the active side. For this reason, the required embedded depth needs to be verified from horizontal force and moment equilibrium.

Excessive deformations sheet pile wall
The active soil wedge can cause deformations to the sheet pile wall if the stiffness is relatively low, at the unsupported area of the wall. If these deformations become too large, it will cause deformations to the concrete walls which is not preferable in terms of the movement of the self-closing flood barrier because the recess in which the gate moves will be more narrow. The profile of the sheet pile wall needs to be verified in terms of sufficient stiffness which relates to the second moment of area which depends on the cross-sectional properties.

Insufficient strength sheet pile wall
The horizontal soil pressure acting on the sheet pile wall causes bending moments in the wall. If the strength is too low, the stresses in the sheet pile wall can exceed the yield stress which can lead to failure. The profile of the sheet pile wall needs to be verified to its section modulus.

Uplift of under water concrete floor
When the under water concrete floor is casted, it experiences upward water pressure of the ground water in the soil underneath the floor, because the ground water table is 1.5 m above the foundation level. This can cause the floor to experience an uplift force. The thickness of the under water concrete floor can be increased in order to obtain sufficient downward force from the additional self-weight making vertical force equilibrium. If the floor requires a too large thickness which is uneconomical, the floor can be anchored with tension piles that take up part of the uplift force.

Use phase

Insufficient bearing capacity of soil (vertical stability)
If vertical loads exceed the bearing capacity of the soil underneath the foundation, the soil underneath the foundation will collapse. For this reason the bearing capacity of the soil needs to be verified. For this the Prandtl and Brinch Hansen method will be used.

Settlement (vertical stability)
A structure can cause the soil underneath the foundation to settle, because of compaction of the soil due to the added weight. This should normally be checked with a calculation. For this design problem, the assumption is that this will not be relevant, because the entire structure is embedded in the subsoil, meaning that before constructing the structure, a certain amount of soil with a self-weight is excavated which is replaced by the structure. So if the replacing weight, which is the weight of the entire gate

system, exceeds this excavated soil weight which initially loaded the subsoil, settlement may need to be checked, because there would be additional weight on the subsoil compared to the initial situation. Otherwise settlement is not likely to occur.

Uplift complete structure
Because the groundwater table is above the bottom level of the foundation, the entire structure can experience an uplift force, because of the upward water pressure. This would be a governing case if a high water event is present, where the water level has not yet reached the signaling water level meaning that the concrete chamber is not filled with water. At this point, the structure is subject to a maximal water pressure coming only from underneath the foundation. However, there will only be uplift if the opposing force working in the opposite direction (downward), which is only the self-weight of the structure, is large enough to resist the upward water pressure.

Overturning (rotational stability)
The structure is subjected to horizontal and vertical forces. The work line of the resulting acting force of all loads should be within the core of the structure, otherwise the eccentricity of the resulting force will exceed the limit where the soil will need to provide tensile stresses which is not allowed, because the subsoil cannot provide tensile resistance. This results in less bearing capacity of the soil to resist the acting moment. This will lead to overturning of the structure. The core of the structure is defined as the area with $\frac{1}{6} \cdot b$ from the midpoint of the structure to the left and right at the bottom of the structure.

Sliding (horizontal stability)
The structure is subjected to horizontal forces which can cause the structure to slide aside in the case of a shallow foundation. The resistant friction force should not be less than these horizontal acting loads. The resisting friction force depends on the material of the structure at the bottom and the soil type. For example, gravel and coarse sand have a larger friction coefficient in comparison with clay or loam.

Piping
A difference in water level (the 'hydraulic gradient') across the flood barrier can cause a flow of groundwater in the sandy soil underneath the foundation. When this flow pours out of the soil, it is called 'seepage'. If the ground water flow has a velocity large enough to extrude sediment particles, internal erosion can occur that develops backward to the high water side of the barrier. This would lead to the formation of 'pipes' underneath the structure in which ground water flows. When these pipes become large enough, the stability of the foundation is undermined. However, with increasing the seepage lengths by placing for examples sheet pile walls, piping can be prevented.

Strength failure
All structural components need to resist the occurring stresses and strains resulting from the external loads acting on the structure. The structural components need to be designed with cross-sectional properties in order to have sufficient strength to limit the stresses. Otherwise structural failure occurs meaning exceedance of bending moment and shear capacity, normal force capacity and/or buckling and lateral buckling capacity. The structural components will be differentiated as such:

- Gate
- Foundation (concrete)
- Connections
- Sheet pile wall

More deepening on the strength verification will follow in the structural design per component.

5.2.2. Governing situation

Two governing situations will be differentiated regarding the structural design for the use phase.

Situation 1: high water event with design water level
This is the situation with the water level which followed from the functional design, which was the design water level of 17.84 m + NAP with a return period of 417 years. In this case the hydraulic gradient across the structure is the highest. This situation is governing for the following failure mechanism verifications:

- Vertical stability
- Horizontal stability
- Rotational stability
- Piping
- Gate design
- Connection design
- Concrete design

Situation 2: high water event with water level just before signaling (non-dominant water level)
This is the situation during a high water event with the water level just before signaling, where the concrete floatation chamber is empty but at the starting point to be filled. At this point the structure is subjected to a maximal water pressure coming only from underneath the foundation. This situation is or could be governing for the following failure mechanism verifications:

- Concrete strength
- Uplift entire structure

The design of the sheet pile wall has a different load case which occurs in the construction phase. The structural design and thus the associated load situation for this phase are tentatively omitted.

5.2.3. Load situations
Permanent loads

Self-weight entire flood barrier
The self-weight of the entire flood barrier can be subdivided into two parts which associated with the main system components.

- Self-weight gate
The self-weight of the gate is not yet precisely quantifiable since the gate design follows in a later step. Tentatively, based on the functional design an estimated gate weight will be used as an initial value for this load contribution. For this, the self-weight will be considered per unit width. An estimate for the required cross-sectional area of the gate is $A_{gate} = 0.10m^2$. The gate is made entirely of steel, which has a specific weight of $\gamma_s = 78.50kN/m^3$. From this follows that the associated estimated gate weight per unit width is $G_{gate} = \rho_s \cdot A_{gate} = 78.50 \cdot 0.10 = 7.85kN/m$.

Parameter	Value	Unit
A_{gate}	0.10	m²
γ_s (steel)	78.5	kN/m³
$\mathbf{G_c} = \gamma_s \cdot A_{gate}$	**7.85**	**kN/m**

- Self-weight concrete
As for the gate, the self-weight of the concrete is not yet precisely quantifiable. Tentatively will be assumed that the concrete chamber has two walls and a floor with a thickness of $t_c = 550mm$. For reinforcement a percentage of 2% of the total concrete volume per unit width will be assumed. The concrete structure has a total height of $h_c = 4750mm$ and a total width of $b_c = 3900mm$.

Parameter	**Value**	**Unit**
Concrete		
$h_{c,wall}$ $t_{c,wall}$ $b_{c,floor}$ $t_{c,floor}$ $A_c = 2.0 \cdot h_{c,wall} \cdot t_{c,wall} + b_{c,floor} \cdot t_{c,floor}$	4.20 0.50 3.90 0.55 6.35	m m m m m²
Steel reinforcement		
ρ (reinforcement percentage = 2 %) $A_s = \rho \cdot A_c$	0.02 0.13	[-] m²
Total		
γ_c (concrete) γ_s (steel) $\mathbf{G_c} = \gamma_c \cdot A_c + \gamma_s \cdot A_s$	25.0 78.5 **168.6**	kN/m³ kN/m³ **kN/m**

Horizontal soil pressure
The total horizontal soil pressure consists of the horizontal effective soil pressure and the groundwater pressure. According to Pascal's law, water pressure is equal in all directions, but this does not apply to soil pressure. Thus, both require separate consideration. The water pressure is elaborated further as part of the variable loads. The horizontal effective soil pressure is linearly related to the vertical effective soil pressure σ_v' by a constant factor K . The vertical effective soil pressure in turn is determined by the following relation.

$$\sigma_v' = \sum_{i=1}^{n} \gamma_{d,i} \cdot d_i + \sum_{j=1}^{m} \gamma_{n,j} \cdot d_j - p \tag{5.1}$$

, in which:

σ_v'	[kN/m²]	=	vertical inter-granular stress (= effective pressure)
$\gamma_{d,i}$	[kN/m³]	=	dry volumetric weight of soil layer i
$\gamma_{n,j}$	[kN/m³]	=	wet volumetric weight of soil layer j
d_i	[m]	=	thickness of soil layer i above the considered plane
n	[-]	=	number of dry layers above the considered plane
m	[-]	=	number of wet layers above the considered plane
p	[kN/m³]	=	water pressure in the considered plane

In summary, the vertical effective soil pressure is determined by multiplying the specific weight of the soil type in each layer with the layer thickness and adding up each stress value per layer until the stress in the considered plane is obtained, which is at the bottom of the structure subjected to the soil pressure. For wet layers, where groundwater is present, the volumetric weight of water needs to be subtracted of the specific weight of the soil.

The soil along the span of the barrier varies, which means that the soil parameters are different for several locations along the span of the barrier. From 'DINOloket', the central gateway to data and information of the Dutch subsoil, there is data available of four relevant locations along the span of the barrier. The locations are shown in Figure 5.5. In Appendix N, soil profiles are reconstructed from

CPT graphs of these four locations and with the help of the Manual Hydraulic Structures (M. Voorendt, 2023) for the classification of the soil types.

Figure 5.5: Locations of retrieved soil data encircled in red and barrier location indicated with the blue line (DINOloket, 2023)

From the reconstructed soil profiles, the horizontal effective soil pressure diagrams over the height of the barrier are created for each of the four locations along the barrier. This is also further elaborated in Appendix N. However, it appears that the diagrams are rather homogeneous and for this reason, the horizontal effective soil pressure is assumed to be increasing linearly constant over the height with an average pressure of $8.3kN/m^2$ from zero to $33.0kN/m^2$ at the construction level for all locations along the span, since the differences between the locations are minimal. The leading horizontal effective soil pressure diagram is also depicted in Figure 5.6.

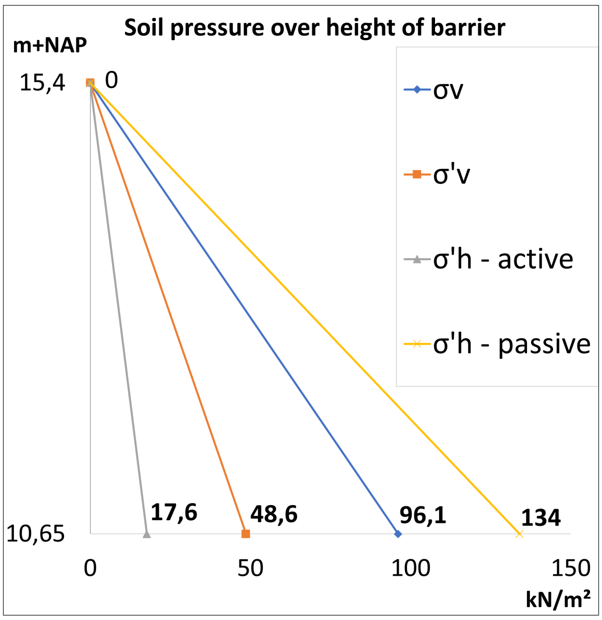


Figure 5.6: Overarching effective soil pressure diagram for entire span (pressure in kN/m^2 on horizontal axis; level in m from construction level to ground level on vertical axis)

Variable loads
The variable loads are solely the high water hydraulic loads. The high water hydraulic load consists of two contributions, the first being the hydrostatic pressure on the structure (both the gate and the concrete) resulting from the still design water level to be retained in case of a high water event and the second being the static contribution of wave loads.

The determination of the magnitude of the pressure distributions are elaborated in Appendix N. Also the simplification of the static wave pressure distribution is justified in Appendix N. The resulting pressure diagrams on the structure are summarised in Figure 5.7 together with the horizontal soil pressures for the most governing situation.

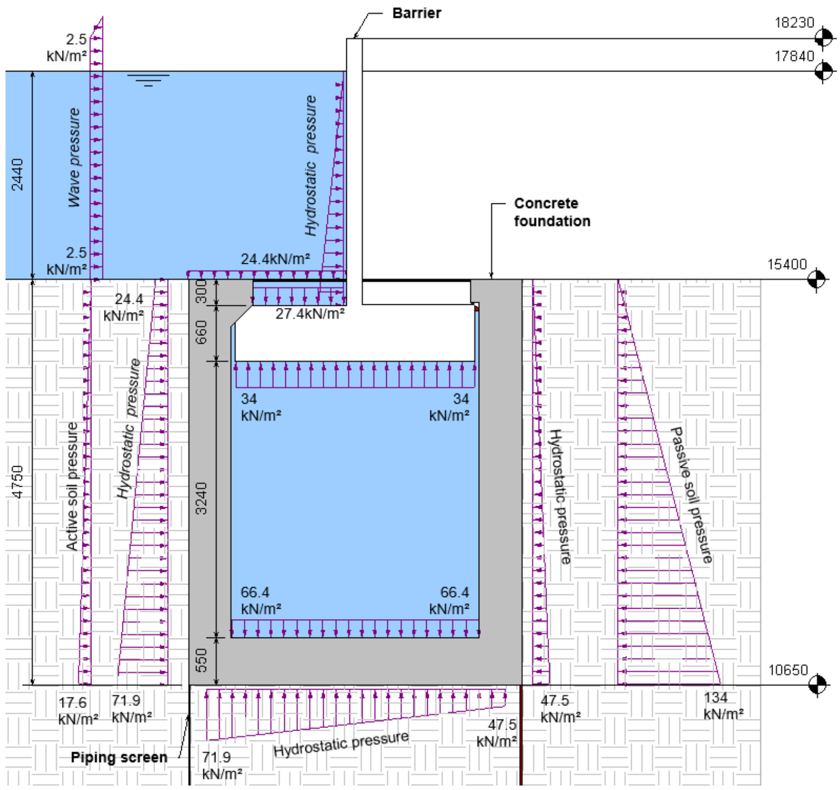


Figure 5.7: Resulting pressure diagrams on the structure resulting from the hydraulic loads

Other loads (not taken into account)
Temperature
Temperature changes of structural components can lead to the development of stresses and deformations. This involves for example cooling due to wind and precipitation, accumulation of warmth due to solar rays or warmth emission during night time. However, the design guide for hydraulic structures states that temperature loads do not require consideration for the evaluation of a hydraulic structure during a high water event, because extreme temperatures are not to be expected during governing circumstances.

Ice
Normally for hydraulic structures such as sluice gates, ice loads require consideration. For the self-closing flood barrier this is not applicable, because, extreme low temperatures are not to be expected during governing circumstances.

Wind load
The computation of the representative wind loads are conform NEN-EN 1991-1-4. However, direct

wind loads are not relevant, since during governing circumstances, wind loads can not be imposed on the barrier. Wind does have an effect on the development of wind generated water waves. This will be taken into account.

Load combinations
Formally, structural failure as indicated in the Design Guide Hydraulic Structures of Rijkswaterstaat (Dutch: WOWK), is the event where a hydraulic structure fails, meaning either insufficient strength of structural components or instability of the entire structure, in combination with a flood and the corresponding consequences, which in specific terms mean exceedance of the water storage capacity in the hinterland and/or erosion of the subsoil which leads to scour holes, stability loss and a continued breach of the structure. From consideration of the Dutch Water Act, with respect to this failure mechanism, a verification involving the combination of several events needs to be performed to determine the probability of failure. However, in practice, after structural failure occurs involving insufficient strength or instability of a hydraulic structure, a flood with its consequences is usually inevitable. For this reason, with designing a hydraulic structure, most of the time solely the initial structural failure is considered instead of the combination of events, meaning that the probability of a flood is set to $P = 1.0$. Thus, structural failure is further considered as solely insufficient strength (SSC) and/or instability (SSS) instead of a combination with the probability of flood consequences. Both structural failure mechanisms are ultimate limit states.

SSC = Strength Structural Components
SSS = Stability Structure and Soil

The Eurocode states requirements for the reliability of structural failure of (hydraulic) structures. For hydraulic structures usually the highest consequence class is considered, which is CC3. The Dutch Water Act also states requirements to the structural failure of hydraulic structures which follows from the standard for dike segments in the Netherlands. In the basis of design, this was already determined for all relevant failure mechanisms. The verification from the Dutch Water Act needs to be performed only in the case where flooding is the consequence of structural failure.

For the design verifications, design values for the loads need to be used. Conform the Eurocode NEN-EN-1990 a distinction is made between permanent, variable and extraordinary loads, in which each load case has its own partial factor. Table 11 of the Design Guide Hydraulic Structures provides these factors which can be used for hydraulic structures in consequence class CC2, but can be adjusted to CC3 by multiplying with a factor $K_{FI} = 1.1$. In Table ... the design values relevant in this design study are summarised.
The effective load effect for each ultimate limit state (ULS) verification, such as stability or bending moment capacity, is a combination of the relevant loads for the particular failure mechanism with corresponding partial factors. The design load effect can be determined by formulas 6.10a and 6.10b from the NEN-EN-1990. Each formula corresponds to the dominance of one of the main load types, namely the self-weight or the hydraulic load. The governing load effect resulting from formula 6.10a or 6.10b is the design load in each verification:

Self-weight dominant	$E_{d,a} = k_{FI} \cdot \gamma_G \cdot G_k + \Psi_0 \cdot S_d$	(NEN-EN 1990 6.10a)
Hydraulic load dominant	$E_{d,b} = k_{FI} \cdot \gamma_G \cdot \xi \cdot G_k + S_d$	(NEN-EN 1990 6.10b)

, in which:

The design hydraulic load S_d does not have a partial load factor since it is determined with an exceedance probability and load statistics in Hydra-NL. The hydraulic pressure diagram as determined earlier will be used to derive the design value of the hydraulic load for the design verifications.

The Design Guide Hydraulic Structures states that the self-weight is dominant if it is over 80% of the total load.

5.3. Stability (overall)

In this section the results of the stability verifications are presented, involving horizontal, vertical, rotational stability, uplift and piping.

The stability verification involves the situation of a high water event with flood consequences. This means that both, a verification based on the Eurocode and the Dutch Water Act need to be performed. However, the Design Guide Hydraulic Structures states that the verification based on the Dutch Water Act is not required, if some conditions are applied which are mentioned in Appendix O. So the Eurocode will be followed with NEN-EN 1990 6.10b for determining the loads, since the hydraulic load is dominant.

The loads acting on the structure in the governing situation for the stability verifications is shown in Figure 5.8. The left image shows all the original forces and in the right image the horizontal ground water pressure and soil pressure are simplified by subtracting one with the other, leaving only the nett pressure in the image.

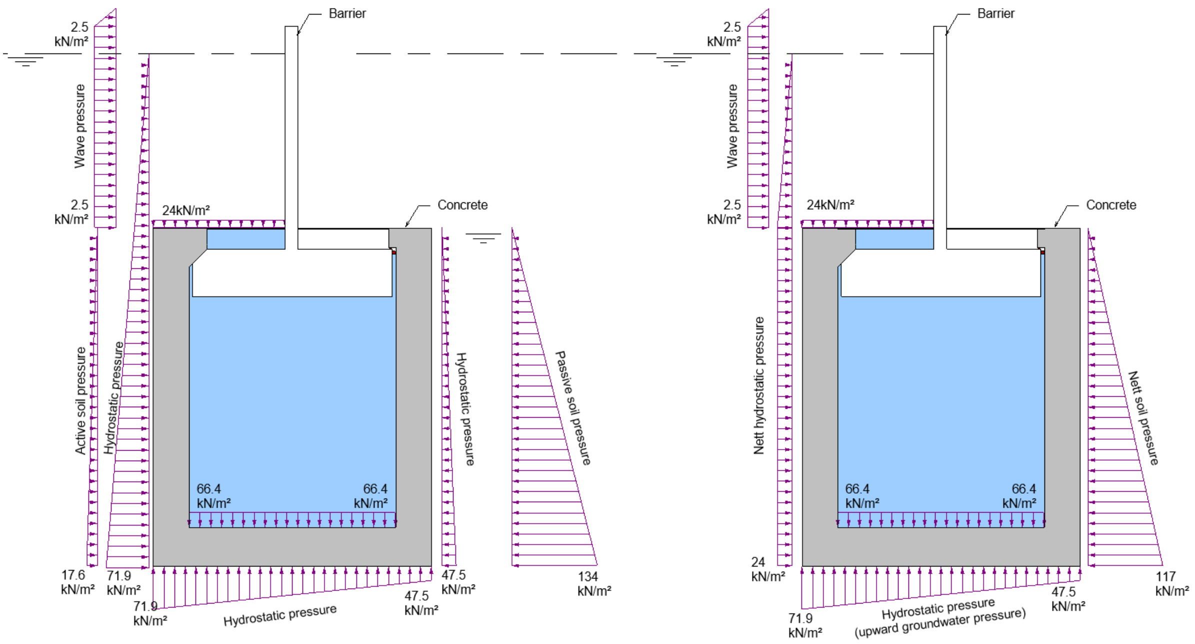


Figure 5.8: Schematic illustration of loads involved in stability verification

5.3.1. Vertical stability

The vertical effective soil stress required to resist the acting loads $\sigma_{k,max}$, should not exceed the maximum bearing capacity of the soil p'_{max}, otherwise the soil will collapse:

$\sigma_{k,max} < p'_{max}$

The collapse of the soil with slip planes is illustrated in Figure 5.9.

Vertical effective soil stress $\sigma_{k,max}$

The vertical effective soil stress per unit span length is determined with:

$$\sigma_{k,max} = \frac{F}{b} + \frac{M}{W} = \frac{\sum V}{b} + \frac{\sum M}{\frac{1}{6} \cdot b^2} \tag{5.2}$$

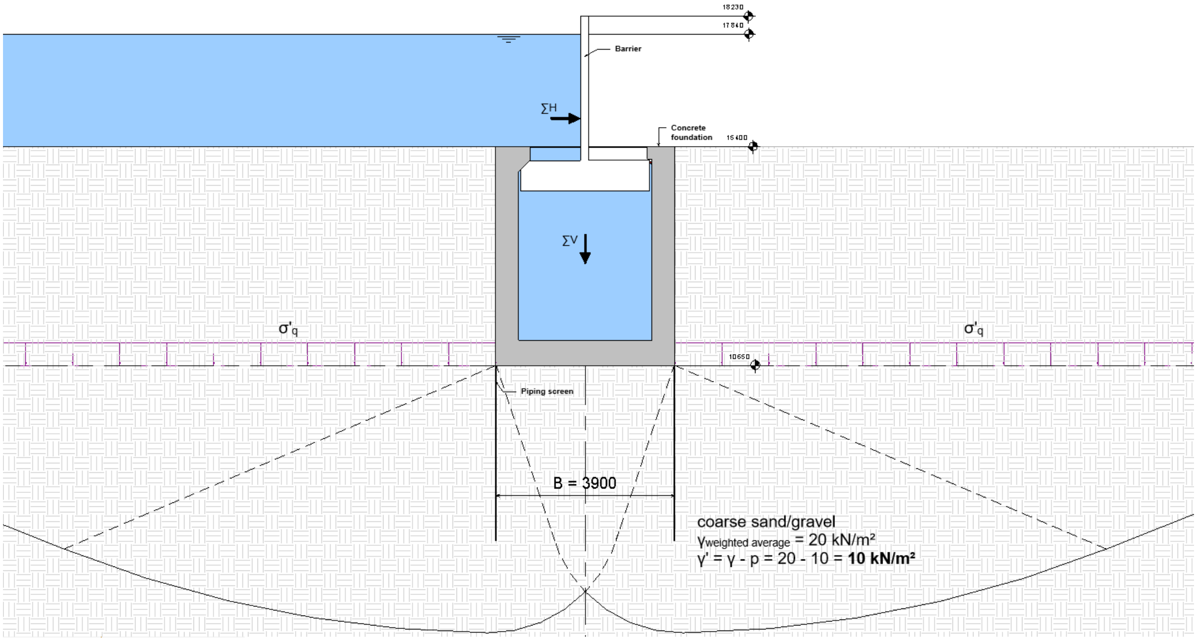


Figure 5.9: Failure mechanism of soil collapse under the structure based on Prandtl's method of theoretical slip planes

Soil bearing capacity p'_{max}

The soil bearing capacity is provided by the soil underneath the self-closing flood barrier. The soil underneath the structure consists of either (coarse) sand or gravel, which have high permeability and behave as drained soil. The soil bearing capacity in drained materials according to the Brinch Hansen method is expressed by:

$$p'_{max,drained} = c' \cdot N_c \cdot s_c \cdot i_c + \sigma'_q \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \tag{5.3}$$

, in which the first term consists of factors denoted with a 'c' which indicates the contribution of cohesion and the second term consists of factors denoted with a 'q' indicating the contribution of effective surcharge pressure, which means loading on the soil surrounding the bottom of the structure. The last and third term consists of factors denoted with γ indicating the contribution of the specific weight of the soil underneath the structure. Expression 5.3 can be simplified to the following expression, because there is no cohesion, $c = 0$:

$$p'_{max,drained} = \sigma'_q \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \tag{5.4}$$

Vertical stability verification

$$\sigma_{k,max} = 54.2 \;\; kN/m^2 \quad < \quad p'_{max} = 8152.1 \;\; kN/m^2$$

5.3.2. Horizontal stability

The horizontal stability is in compliance if the sum of the horizontal forces exceed the friction force between the structure and the soil. The ratio between the total horizontal force and the vertical force relates to the friction between the bottom of the foundation and the soil. The friction for casted concrete on coarse sand should be around 0.55 (Manual Hydraulic Structures, 2023) or lower to prevent sliding. The resistance to sliding is assumed to be sufficient, observing if the following friction ratio does not exceed f.

$$\frac{\sum H}{\sum V} < f \tag{5.5}$$

The nett horizontal force on the foundation, i.e. the sum of the horizontal forces acting on the foundation, results in a force directed to the left hand side which is towards the opposite side of the main hydraulic loads. This is because of the passive soil pressure acting on the structure from the right hand side. This maximum pressure exceed the main loads which in practice can not occur. The soil mobilises as much as is required in order to be in equilibrium with the horizontal loads. This equilibrium occurs where the sum of the horizontal forces is equal to the friction force which is the sum of the vertical forces multiplied with the friction coefficient:

$$\sum H = f \cdot \sum V = 0.55 \cdot 161.7 = 88.94kN$$

The nett horizontal force on the foundation thus needs to be 88.94 kN. This means that in order for this horizontal force equilibrium to occur, the soil needs to mobilise as much that the nett soil pressure on the structure develops to have a resultant force which is 248.6 kN which provides the horizontal stability and is named H4 in Figure 5.10. This force is important to take into account for the rotational stability check in the next section. The loads on the structure for this verification are summarised in Figure 5.10

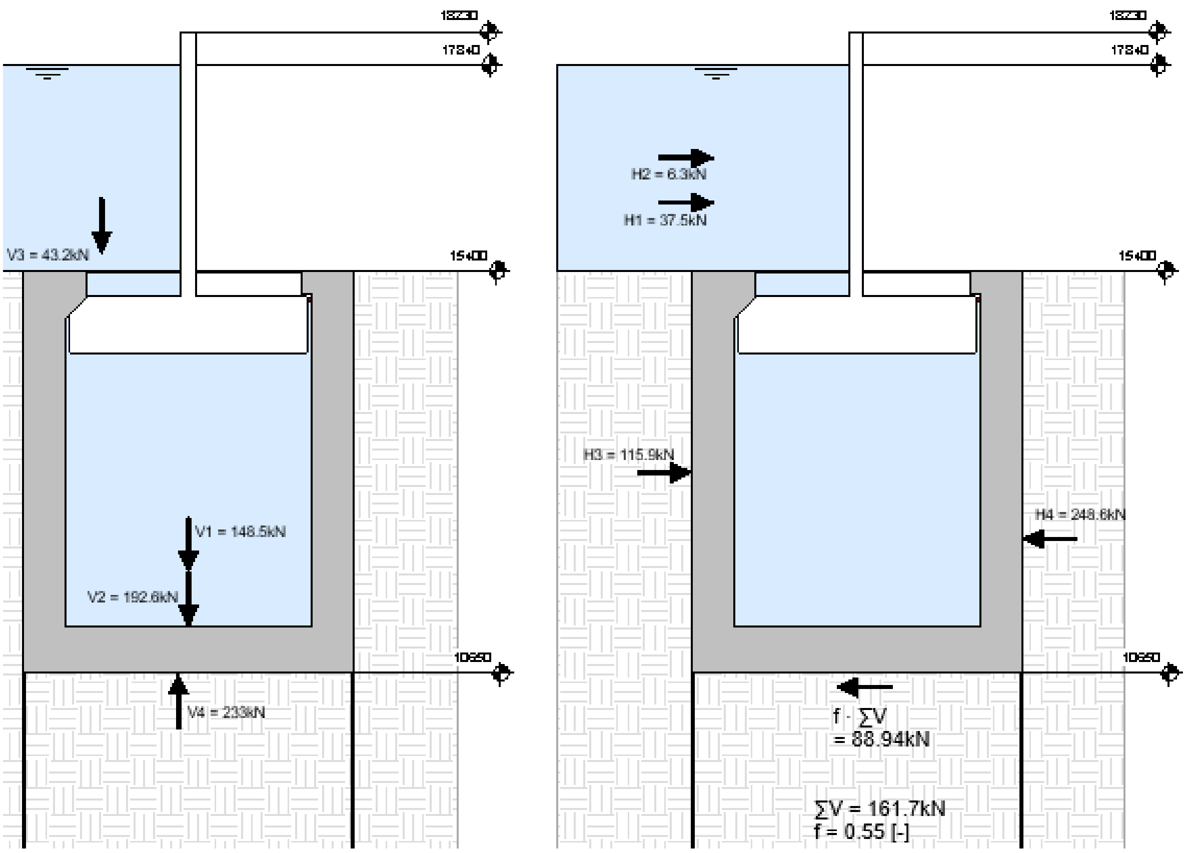


Figure 5.10: Indication of all the loads acting on the structure for horizontal stability

Thus, the structure is horizontally stable, because there is sufficient counteracting horizontal passive soil pressure to make sufficient equilibrium.

5.3.3. Rotational stability

To ensure the rotational stability, the following requirement must be met:

$$e_R = \frac{\sum M}{\sum V} \leq \frac{b}{6} \tag{5.6}$$

where :	e_R	[m]	=	distance from middle of the structure to the intersection point of resultant force and the bottom line of structure
	$\sum V$	[kN]	=	the sum of vertical forces
	$\sum M$	[kNm]	=	the sum of acting moments around overturning point
	b	[m]	=	width of structure

The rotational stability verification is summarised in Figure 5.11. Note horizontal force H4 from the horizontal stability verification which represents the nett horizontal soil pressure force on the structure, which means the sum of the passive and active soil pressure force.

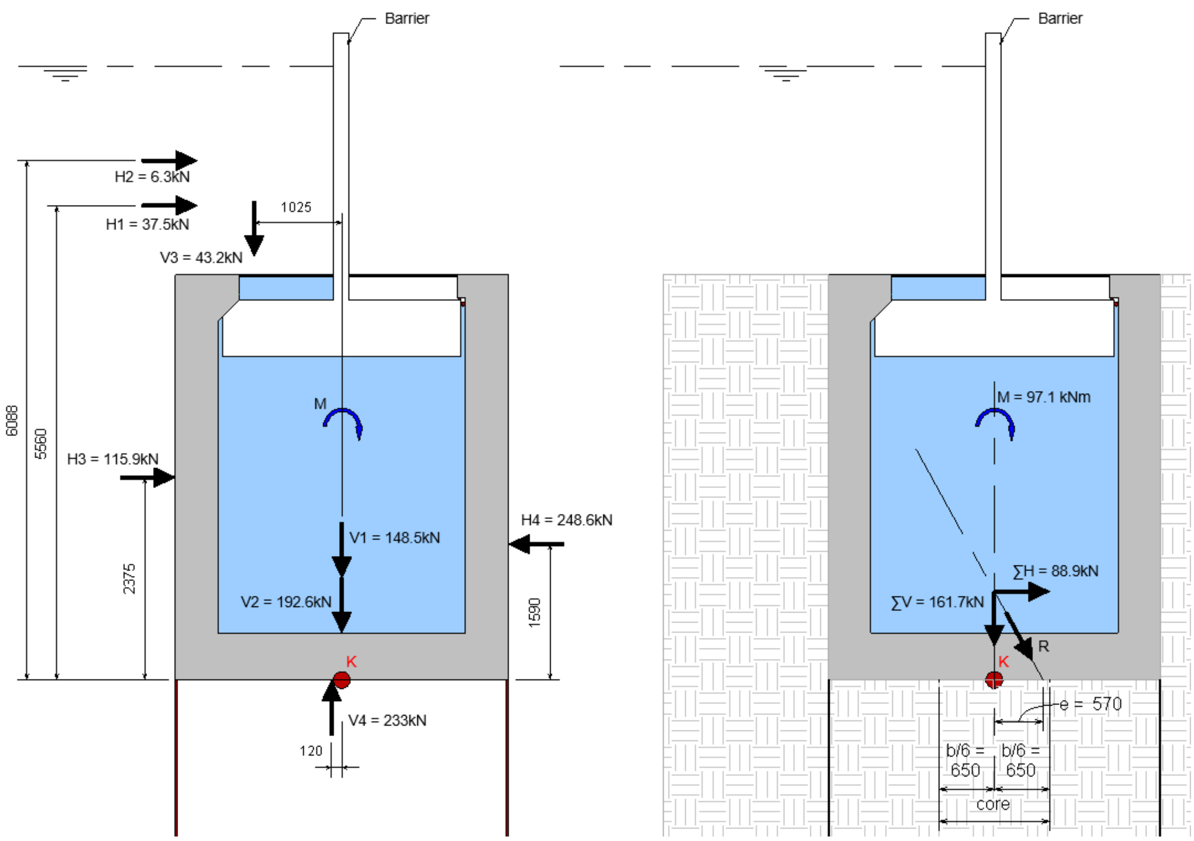


Figure 5.11: Indication of all the loads acting on the structure for rotational stability

In Figure 5.11 it can be observed that the rotational stability is in compliance which is numerically shown here:

$$e_R = \frac{\sum M}{\sum V} = \frac{97.1kNm}{161.7kN} = 0.57m \leq \frac{b}{6} = 0.65m$$

5.3.4. Uplift entire structure

The governing case is an empty floatation chamber and a water level at NAP + 13.65 m in the Meuse, which is approximately the water level where the floatation chamber starts to fill, leading to a saturated soil and a similar ground water level. The ground water pressure at the bottom of the foundation (NAP + 10.65 m) will be 30.0 kN/m². The width of the entire structure is 3.9 m. The upward force per unit span length is 117 kN/m as determined earlier. The design value of the weight of the structure in rest comprises of the gate weight and the weight of the concrete, which is in total 158 kN/m. The upward water force is 117 kN/m. Observing the verification below, uplift will not occur:

$$Q_{G,Ed} = 158 \ \ kN/m \quad > \quad F_{upward} = 117 \ \ kN/m$$

Note that the loads are per unit meter span length in longitudinal direction.

5.3.5. Piping preliminary

There are two types of internal backward erosion, namely:

- piping: formation of erosion pipes under a structure
- outflanking: formation of erosion pipes around a structure

Outflanking

Outflanking is not in question in this case, because on one end of the dike section, the popular touristic restaurant 'Alt Arce' is located where it is assumed that already seepage screen are present to prevent outflanking of the foundation of the restaurant. Besides, the soil of the area of that southern end of the dike section is comprised of clay in the first 3 m which acts as impermeable soil layer causing the avoidance of entry or exit points for piping. The other end of the dike section is in fact the begin of the next dike section with the following self-closing flood barrier. The design of this dike section is omitted in this thesis. For this reason, only piping is considered, provided that the structural transitions form adequate connections.

Piping

Piping is considered negligible if the length of one of the seepage screens is more than twice the water head difference over the structure in a situation corresponding to the signaling water level in the Meuse (van Bree, 2015). The maximum water head difference in case of the design situation with high water: $\Delta H = 2.44$ m. This means that a minimal seepage path length is required of $L = 4.88m$. The embedded depth of the structure is already 4.75 m, because the foundation level is at 10.65 m NAP. The total seepage path length is then equal to 10.8m, which is far above the required length, which means no seepage screens are required.

5.4. Gate design

In this section, the gate will be designed on the failure mechanisms related to stability and to strength for the most critical situation. The stability involves the force equilibrium of the statically determined structure. The design on strength involves the check of the stress in each structural subcomponent of the gate which should not exceed the yield stress.

5.4.1. Critical situation and failure mechanisms

The critical situation for the failure mechanisms instability and insufficient strength is the extreme event with high water of NAP + 17.84 m. The critical situation is schematised in Figure 5.12.

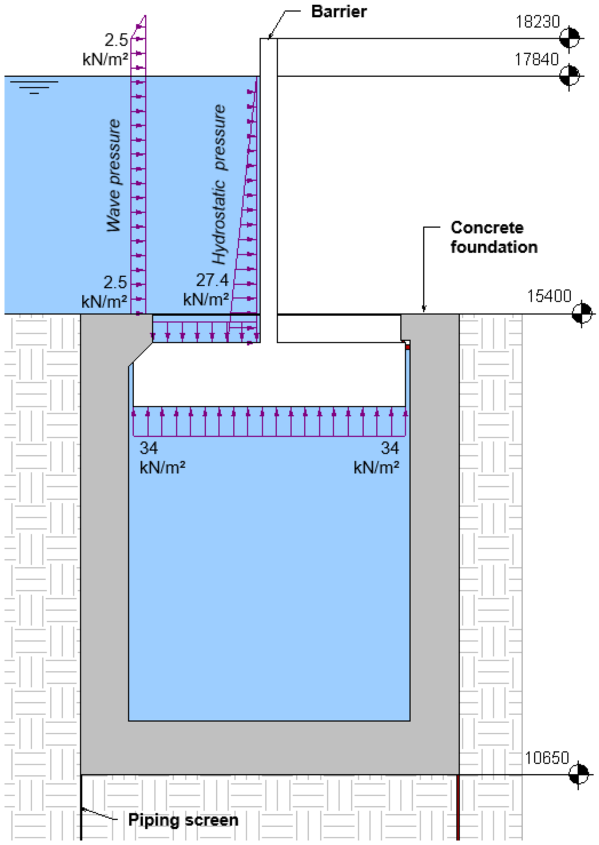


Figure 5.12: Critical situation for the failure mechanisms instability and insufficient strength of the gate

Failure mechanism 1: instability of the gate in retaining position
The failure mechanism involving the instability of the gate in retaining position is illustrated in Figure 5.13. This phenomenon occurs if the gate is not locked in. This allows the gate to tilt and partially float out of the chamber because of the acting hydraulic loads from underneath the gate and against the retaining side of the gate.

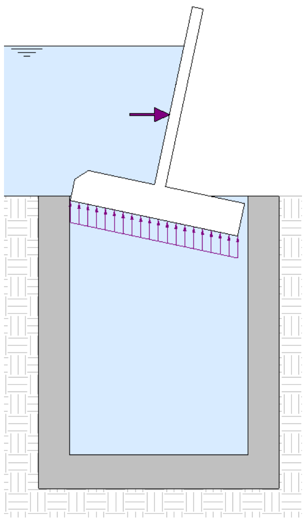


Figure 5.13: Illustration of instability of the gate during the retaining function

Failure mechanism 2: insufficient strength of the gate components
The second failure mechanism of the gate is the insufficient strength of the components of the gate. This can be expressed by the criterion of Von Mises for general plane stresses which states that the yield stress in all fibres of every cross-section should not be exceeded:

$$\sqrt{\sigma_x^2 + 3 \cdot \tau^2} \leq f_y \tag{5.7}$$

where :	σ_x	[N/mm²]	=	stress in normal direction
	τ	[N/mm²]	=	shear stress
	f_y	[N/mm²]	=	yield stress

Note that P.16 represents a simplified verification of plane stress in a cross-section where the normal stress only has one direction.

5.4.2. Geometry modeling

In Figure 5.14 a model of the gate is shown in which the various components are indicated that are required to provide sufficient strength and stiffness and an efficient distribution of loads. The main components of the gate are firstly the skin plate as this is the component that directly retains the water and secondly the floater box girder as this ensures the floatability of the gate. In order to distribute the acting loads efficiently from the skin plate to the concrete structure, the gate requires horizontal and vertical stiffeners, which act as girders and columns.

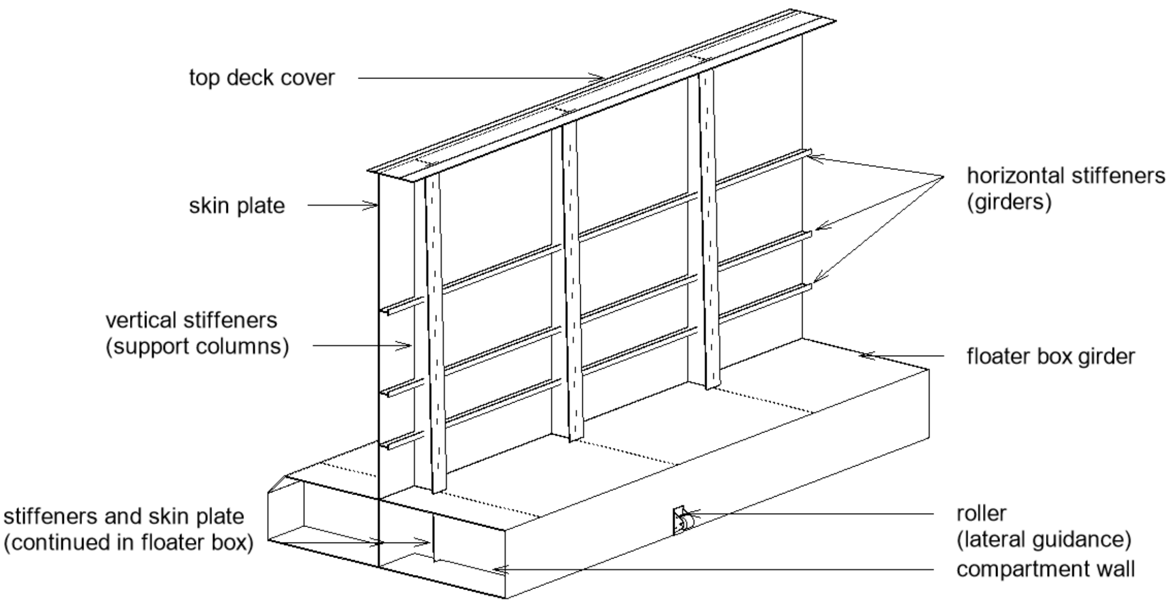
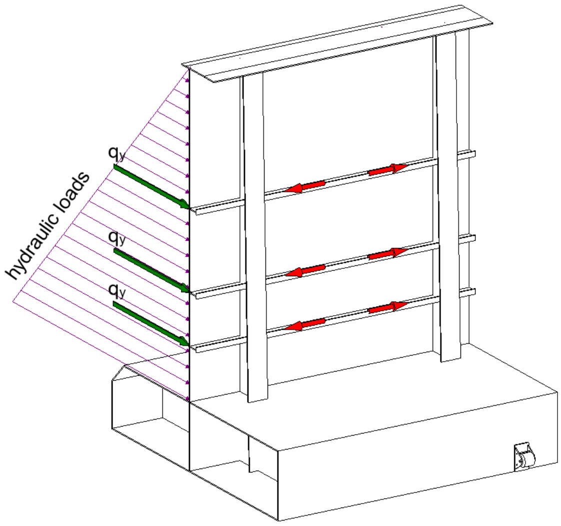
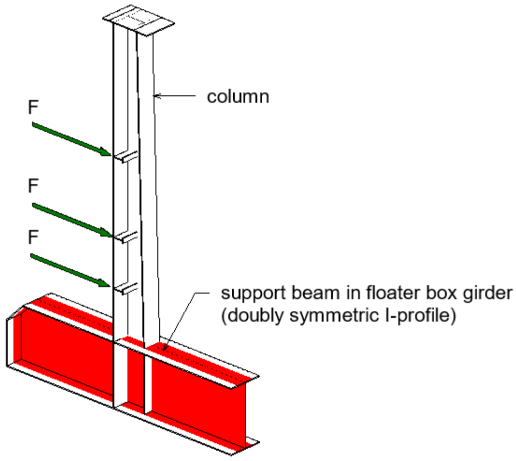


Figure 5.14: 3D model of the gate

The girders increase the bending stiffness, divide the total hydraulic load and transfer it to the columns. This is illustrated in Figure 5.15a. The columns subdivide the total gate span in subspans, which can be seen in Figure 5.15a, and the columns transfer the loads to the compartment walls inside the floater box, which from a functional point of view are present to compartment the floater box girder to increase robustness of the system, but from a structural point of view are capable to also act as support beams for the columns to transfer the loads to. This is illustrated in Figure 5.15b.



(a) Illustration of load transfer from the skin plate to the girders and in turn to the columns



(b) Indication of support beam in red with the compartment walls as the web and an effective width of the floater box as flanges

Figure 5.15: Load directing model

The initial dimensions are determined based on rules of thumb from Erbisti. The results are summarised in Figure 5.16. Note that the spacing of the horizontal stiffening girders is related to equally dividing the total load.

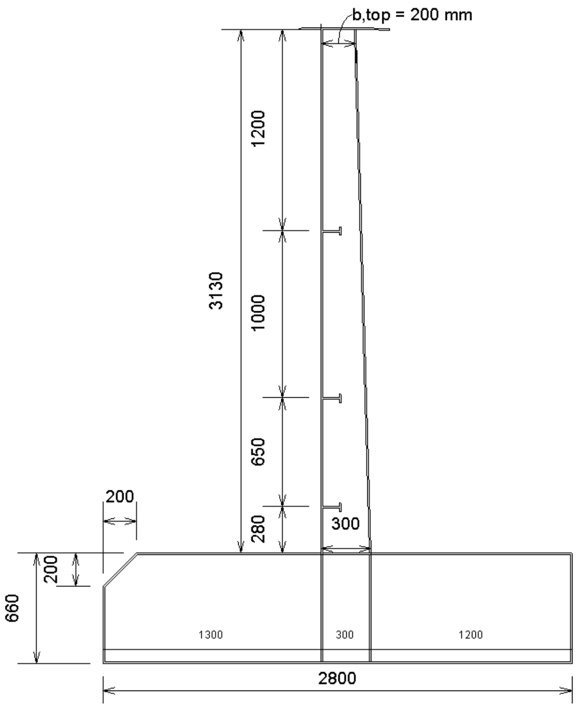


Figure 5.16: Initial dimensions of gate

Lastly, the girders in the floater box, transfer the loads to the flanges of the floater box girder that transfer the loads in turn to the concrete structure.

5.4.3. Force equilibrium of statically determined structure

To prevent instability of the gate in retaining position, the top of the concrete structure is executed with concrete support blocks which lock the gate in. In the functional design, this was already briefly covered. The scheme of the statically determined structure of the gate as a free body diagram is shown in Figure 5.17 with the acting loads indicated in red. The resulting support reactions on the concrete structure ensuring horizontal, vertical and moment equilibrium are indicated in green in Figure 5.17. Important to notice here is that the left concrete support does not provide horizontal support, which results in schematising it as a roller support. The concrete support on the right hand side provides both vertical and horizontal support via the seal connections. It is schematised as two supports to relate it to the locations of the seals. The gate is exposed to sufficient upward pressure to be locked in.

As was described in the functional design, the right hand side of the gate, which is the dry side is sealed to ensure water tightness. The left hand side of the concrete support block has a bevel configuration in order to increase the compression area.

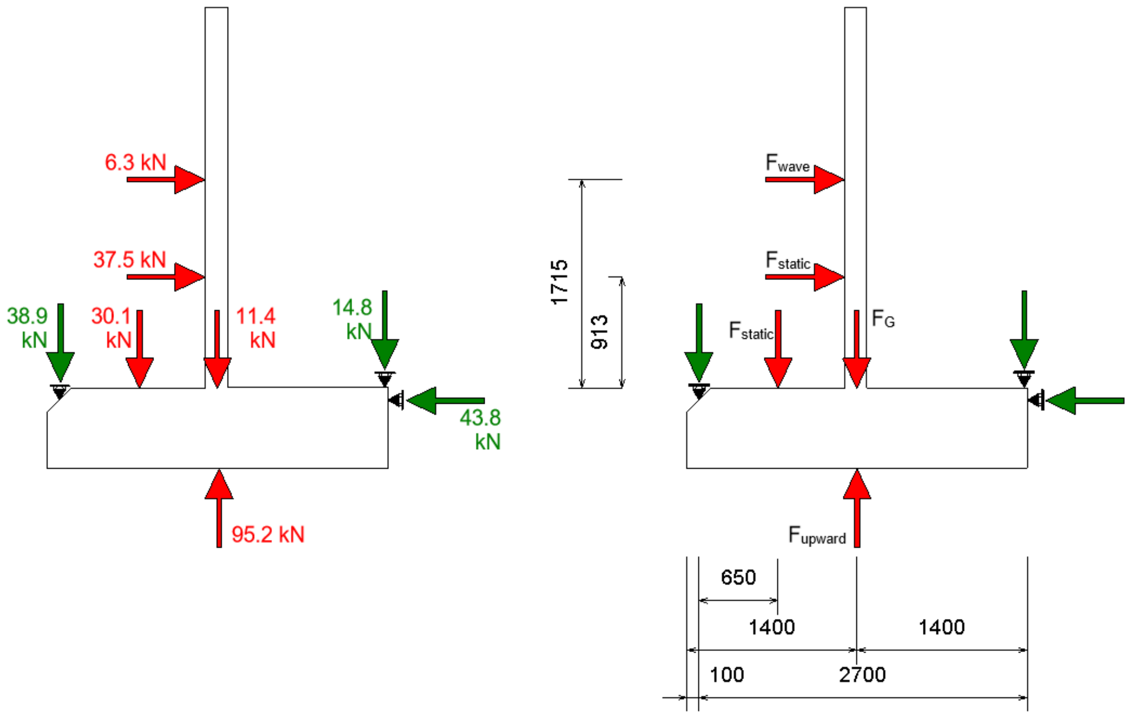
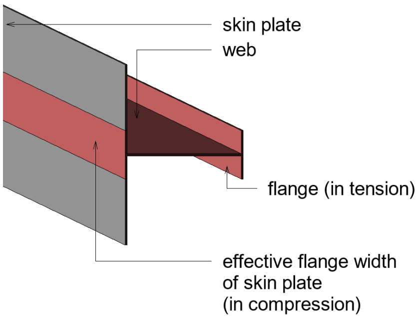


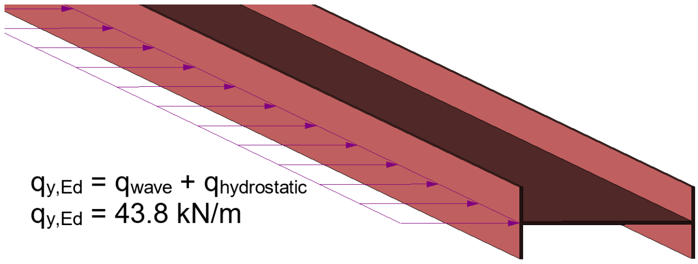
Figure 5.17: Statically determined structure in equilibrium

5.4.4. Design of stiffening girder on strength

In Figure 5.18a the girder is shown for which the design on strength will be made. The failure mechanism involves the exceedance of the yield stress in the governing cross-sections. This results in the following limit state functions. This girder has a span length of 2 m and is supported by columns as illustrated in Figure 5.14 and Figure 5.15a. In Figure 5.18b the acting loads on the girder are shown, which are equal for all three girders. The determination of the design loads on the girder can be found in Appendix P. Important to notice here is that the self-weight of the girder in this check is omitted because of its relative insignificance in magnitude compared to the hydraulic loads.



(a) Indication of girder



(b) Loads on gate

Figure 5.18: Model of girder

The associated structural mechanics scheme is as the following in Figure 5.19. From the verifications according to the Eurocode given in Appendix P the final cross-section for girder type A which is acceptable is shown in Figure 5.20 with given dimensions. The utilisation percentage for this girder is 47%.

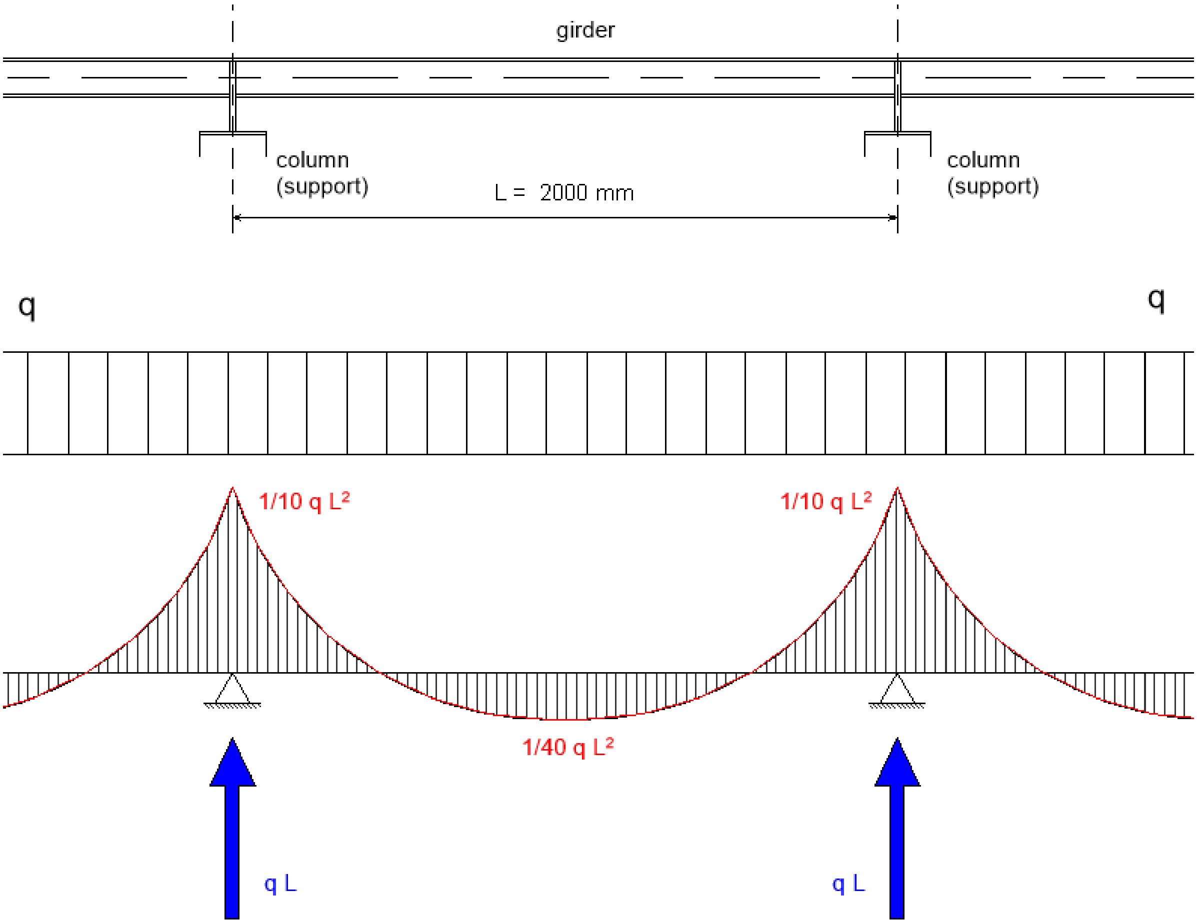


Figure 5.19: Structural mechanics scheme of girder

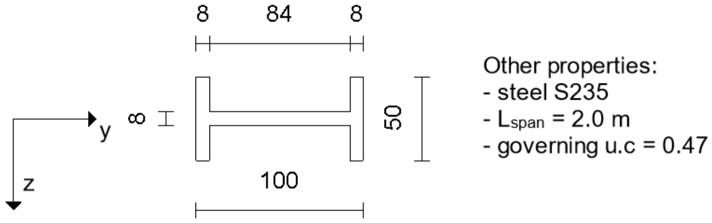
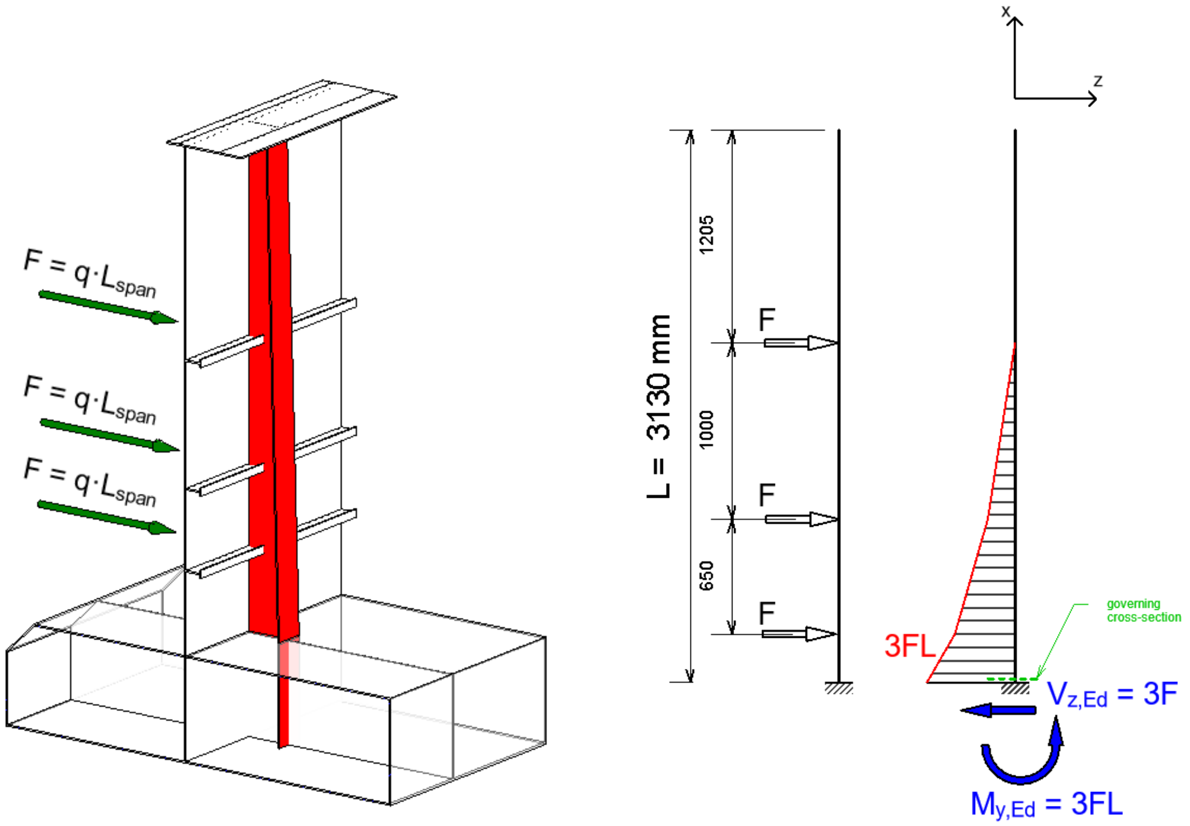


Figure 5.20: Cross-section girder

5.4.5. Design of column on strength

In Figure 5.21a the column is shown for which the design on strength will be made. The failure mechanism involves the exceedance of the yield stress in the governing cross-sections. In Figure 5.21b the acting loads on the governing column are shown, which are equal since they are the support reactions of each girder. Also the associated structural mechanics scheme is as shown in Figure 5.21b. The determination of the design loads can be found in Appendix P. For the governing cross-section, indicated in green in 5.15a the design is made.



(a) Indication of column (b) Loads on column

Figure 5.21: Model of column

From the verifications according to the Eurocode given in Appendix P the final cross-section for the column which is acceptable is shown in Figure 5.22 with given dimensions. The utilisation factor for the column is 71%.

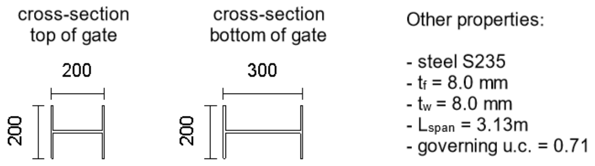


Figure 5.22: Cross-section gate

5.4.6. Design of support beam on strength

Lastly the support beam is designed on strength which is shown in Figure 5.23. The critical situation is already shown in Figure 5.12 and the column beam connection is shown in Figure 5.15b. From this the structural mechanics scheme can be derived as is shown in Figure 5.24. The distributed loads result from the water pressures on the beam over a span length of 2 m. The moment and normal force are reaction loads from the column being transferred to the support beam. Lastly, the vertical force is the resultant of the self-weight of the gate of a section with a span of 2 m. The calculation of the stresses in the cross-section is done with the help of Matrixframe and the results of this are to be found in Appendix P. The support beam does not exceed the yield stress in any cross-section.

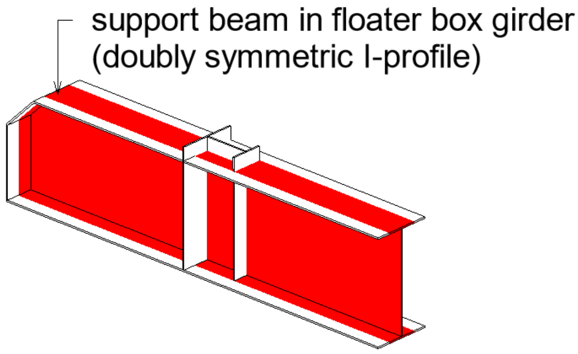


Figure 5.23: Indication of support beam

Support reactions

The resulting support loads are shown in Figure 5.24. Note that the loads in Figure 5.24 are calculated for a span of 2 m and that the loads calculated for the stability of the statically determined structure as shown in Figure 5.17 are calculated per m span. If compared with the same span length, it can be observed that the calculated values are more or less equal, which should be the case. The difference is not significantly large and is probably caused by rounding errors.

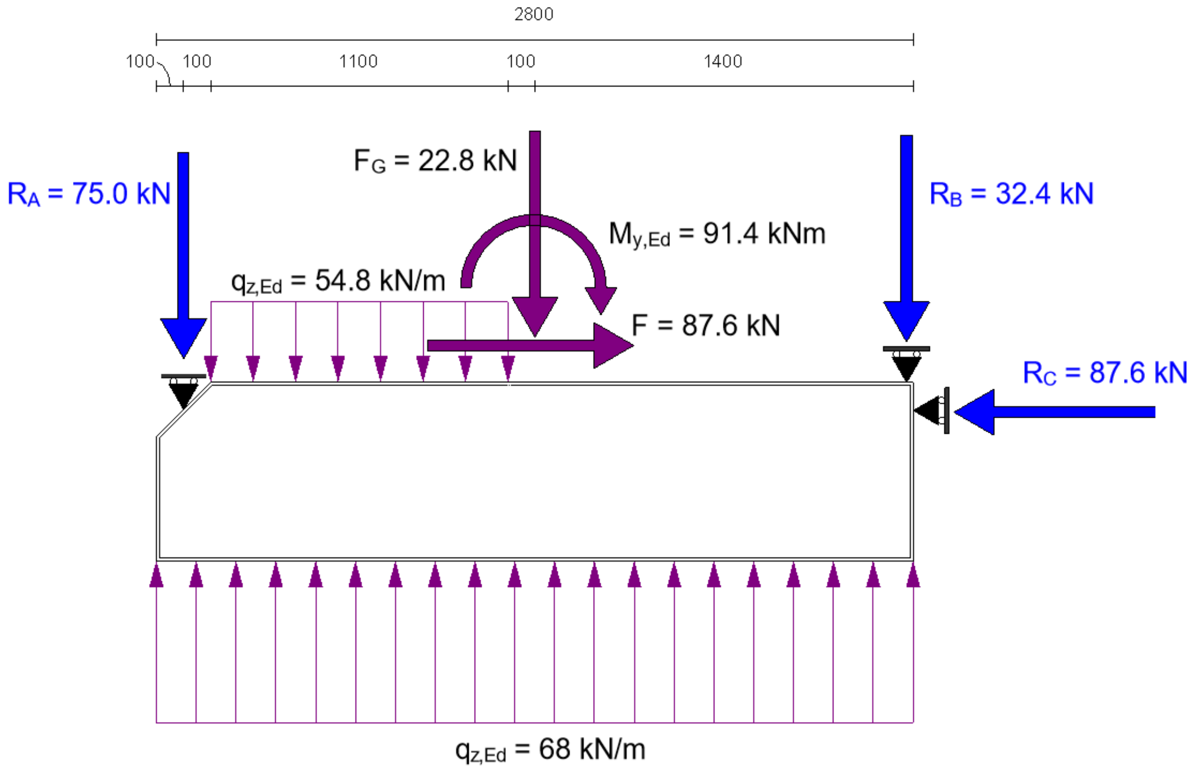


Figure 5.24: Structural mechanics scheme of loads on support beam

The resulting cross-section is shown in Figure 5.25. The governing unity check is u.c = 0.24. This is highly overdimensioned. However, the dimensions are not chosen from a structural point of view but rather a functional point of view. The height of the floater box of h = 660 mm is required for the floatibility and the compartment walls are placed to prevent total failure if there is a leakage in one compartment. The thickness of the plate elements comprising this support beam could be reduced. However, the risk for web shear buckling will be larger leading to more measures such as welding on stiffeners which is labour intensive. One could even argue that using the effective girders within the floater box to use as supports directing the loads to the concrete is actually cost-effective, because the material is already present.

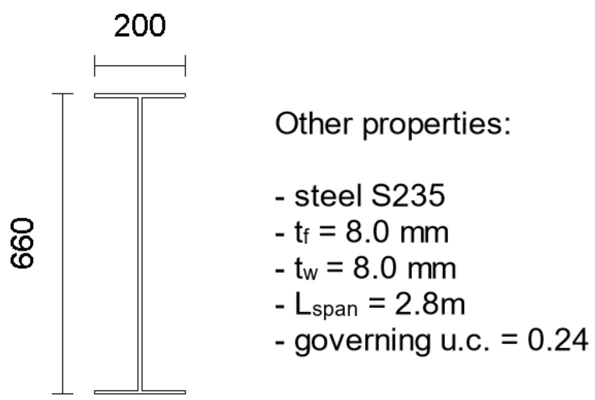


Figure 5.25: Cross-section of support beam

5.4.7. Check of (skin) plate bending stresses

The check of the the plate bending stress in the skin plate is checked with the help of the software DIANA. For the check, the most governing plate module is used with the largest pressure acting on it. The plate has steel quality S235, a thickness of t = 8 mm and for the poisson ratio a value of $\nu = 0.3$ is used. The full elaboration on this is given in Appendix P. The plate is modeled with fixed supports on all edges. In practice this is not entirely true, because the plate modules are only one sided supported by the stiffening girders and columns. However, because of the fact that the hydraulic loads are uniform over the major span, the plate behaves structurally similar to a plate which is fully fixed on all edges. For this reason, the results of a structural analysis for a plate with four fixed edge supports are used cautiously as an indication for the bending stresses in the most governing plate. The results are shown in Figure 5.26 where it can be observed in the legends with colour scales that all maximally occurring stresses are substantially under the yield stress.

Output of results from structural linear elastic analysis of plate bending stresses in DIANA FEA

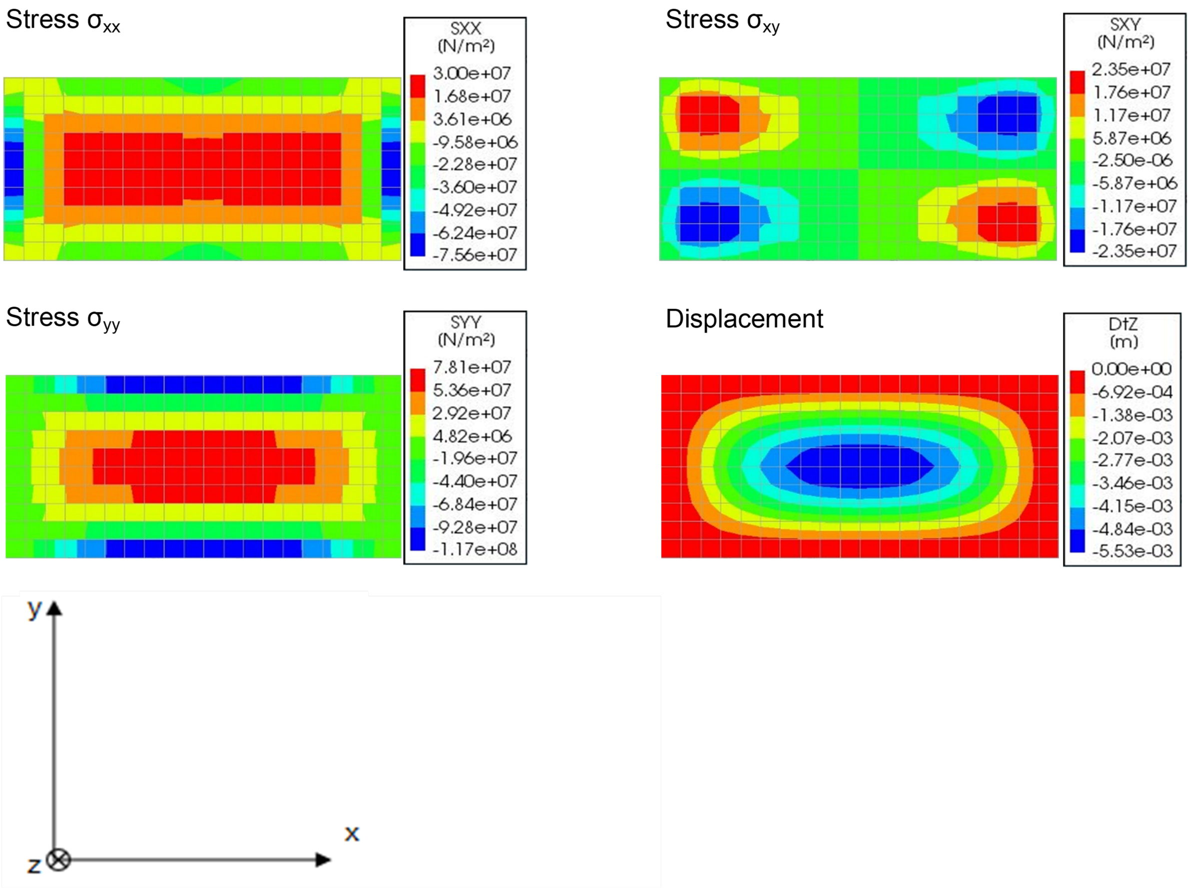


Figure 5.26: Resulting diagrams of DIANA FEA structural linear elastic analysis of bending stresses in governing skin plate module

6 Generalisation and Discussion

6.1. Generalisation

In this section the design is generalised by qualitatively describing how the design process and the potential outcome would change when the location of the barrier would be changed to a lower river region such as in Dordrecht or a coastal area such as in the province of Zeeland.

In the case of a lower river region such as at Dordrecht, the following site characteristics and boundary conditions apply:

- urban area which is more cultivised and has more buildings;
  - there is less consideration needed to be given to the townscape and visibility;
  - the location for such a barrier is more along quays;
  - more consideration has to be given to present foundations and pipe works;
  - the barrier can be loaded with traffic loads;
- proportion of significant wave height with respect to hydraulic loads is higher;
- the soil consists more of clay;
- the groundwater tables are closer to the ground surface level
- the water storage capacity of the hinterland is more governing than erosion;
- the standard for floods is 30 times stricter than for Arcen

This brief list of characteristics and boundary conditions is already majorly different from Arcen, which can lead to the following design choices:

- the barrier is executed with support structures above ground surface level
  - intermediate supports can be helpful to take up wave loads
  - the action line of the nett hydraulic force is more towards the top of the gate
  - the barrier type is different, e.g. air filled bellows barrier or a flap gate which both have a lower foundation level and is better integrable with a quay in terms of additional functions
- a shallow foundation is most likely not possible
- the self-closing principle can still be a floatation mechanism
- the structure should be well resistant to dynamic load effects

If for a location in Dordrecht at quays along for example the rivers Oude Maas or Beneden Merwede a movable hydraulic structure needed to be constructed that integrates in the area where there is less space or not suitable to build an earthen dike, a self-closing flood barrier would be an option to consider following the same design process as in this report. However, the design would most likely majorly differ from the design in this thesis as was briefly outlined. For example the solution would be more similar to the self-closing flood barrier from Hyflo Bv in Spakenburg, but maybe on a larger scale. Another option would be a totally other gate concept such as an air filled bellows barrier or a flap gate such as the to be build Vlotterkering in Steyl-Maashoek.

For a coastal area such as in Zeeland which is far less urbanised than Dordrecht:

- similar consideration needs to be given to the townscape and visibility of the structure as with Arcen;
- the location for such a barrier is more along small waterways instead of large water bodies;
  - Zeeland is mainly protected with earthen dikes, since there is sufficient available space
  - the result of this is that the structure will most likely be smaller
- proportion of significant wave height with respect to hydraulic loads is higher;
- the soil consists more of clay;
- the structure needs to be resistant to brackish or salt water by corrosion protection
- the groundwater tables are closer to the ground surface level
- the standard for floods is 10 times stricter than for Arcen

In conclusion, a structure as presented in this report will not be suitable in Zeeland. The expectation is that, when considering a similar structure, the boundary conditions in Zeeland will not result in the same sized self-closing flood barrier as in Arcen. For example it could be used within city-centres but on a smaller scale. On the other hand, it may be a solution to integrate within earthen dikes as an alternative for dike heightening, if for example dike heightening contravenes with spatial quality or townscape just as with Arcen. In this configuration, the structure directly functions as a dike reinforcement within the dike as well. Very important with considering such a solution direction is the cost-efficiency of the structure.

6.2. Discussion

6.2.1. Selection barrier type and drive mechanism

Barrier type

Reflecting on the selection of the barrier type, the flap gate may have been an interesting solution to consider further in the design process as well for comparison with the result of the flat gate. The reason for this is because the flat gate selection results in a largely dimensioned floater. This is partly because the required buoyancy depends on the total gate weight and additional aspects. However for a flap gate, a part of the self-weight of the gate is directed to the hinge, which reduces the required buoyancy. Furthermore, because of the rotational mechanism with a flap gate, the foundation depth reduces in contrast with the flat gate where the entire gate height needs to be submerged in the underground structure to comply to townscape obstructing requirements. This also results in a heavier underground structure.

It means that the gap between the quality with respect to space usage and effect on the spatial quality of the two gate types is closing. Besides, with a flat gate, the rotational stability of the embedded structure is not easily ensured since the unity check is very high. More on this can be found in Section 5.3.3. A flap gate comes automatically with a larger width which may reduce these stability issues. However, for the location of the self-closing flood barrier and in particular the chosen dike section for the design, a rotational mechanism that is characteristic for a flap gate still results in a large portion of the gardens being taken up that must be kept clear for the gate to move freely. The effect of the barrier being there is much stronger and perhaps undesirable for the residents. An implementation in the area of a flap gate structure needs further research with respect to the viewpoint of residents in the area, with the aid of, for example surveys on such plans. From a technical point of view there are opportunities in which the flap gate may be more efficient than the flat gate.

Drive mechanism
Looking at the selection of the drive mechanism, the ease of construction was scored the highest for a flat gate driven by buoyancy. However, it has been proven that the construction is not as easy as was considered in the MCA. For example, the amount of mechanical or small steel parts are proven to be required more. This means that in relation to the ease of construction the gap between applying a cylinder or a buoyancy driven is closing, which makes it interesting to consider a solution direction with a cylinder. However, the autonomous feature of a buoyancy driven system is still more self-evident than with a cylinder.

Furthermore, one may argue that a cylinder driven system is more reliable since it is widely applied to hydraulic structures on even a larger scale. Even though the latter is true, a buoyancy driven system still was scored better on sustainability and even more important on the complexity and the operational reliability. The complexity involves the amount of space usage and the ease of construction. The main reason that a buoyancy driven system comes out of the evaluation better is because it does not require machine rooms that occupy additional space and require more installation works. The operational reliability involves on the one hand the availability of a back-up drive and its simplicity of deployment. On the other hand it includes the ease of operation which relates to number of processes involved to close the gate. For a buoyancy driven system, the operation is easier and does not have to rely on mechanical processes. Also the back-up drive is high in robustness.

Costs
In the selection of the barrier type and the drive mechanism the costs are not considered. Including this in either the MCA or in a follow up step with plotting a value-cost diagram, may have resulted in another outcome. By defining the scope of the project and prioritising on other aspects such as the functional and the structural design, it was chosen not to include a rough cost estimate in the selection process because the outcome would not be reliable and would therefore negate the added value. Only a good estimate would have been valuable, but there is chosen in this project to exclude a well-supported cost-analysis with respect to the scope of the project.

6.2.2. Erosion protection
The design in this thesis focuses on dike section 2 in the project area. This involves a location which crosses private gardens of residents of Arcen. Therefore, with the determination of the hydraulic load level and the maximum critical overtopping discharge no erosion protection was considered. The reason for this is because the goal is to limit the effect on the spatial quality. This resulted in a relatively high retaining level of the structure. However, for other dike sections in the area, this would not necessarily be the same solution direction. For example, the location of the barrier at dike section 1 is beyond the area of the private gardens near a walking path. For this location, erosion protection is applicable or already present as pavement, which results in a higher critical overtopping discharge that is allowed than with the case for dike section 2. This means that also the retaining level would be lower for dike section 1. Also, dike section 2 that is elaborated on in this thesis will have a significantly different design, particularly with respect to the height and embedded depth in the ground, if for example erosion protection would have been applied. The resulting retaining height will be lower, since more overtopping discharge is allowed. It is only, then, that the question arises whether that is an appropriate solution direction, as the location of the barrier is in a private garden.

Knowing that other dike sections are not limited to apply erosion protection, it may be interesting to investigate a design with distinctive retaining levels between the dike sections. In other words, each dike section with a self-closing flood barrier can be considered as a separate structure with separate boundary conditions. This may lead to a site specific solution direction for the entire dike segment, which is more efficient.

6.2.3. Reliability of closure

The design on the reliability of closure is based on the failure probability requirement regarding the failure mechanism of not closing. This requirement is derived from the standard for the dike segment in this project. From this requirement and mainly the probability of inflow the requirement for the closing mechanism itself follows. With a customised fault tree, it is proven that a self-closing flood barrier with the boundary conditions for this project location is considered feasible, seeing that the maximum individual failure probabilities of the functional components represent a realistic order of magnitude. However, it is important to notice that the individual failure probabilities do not represent actual values. Complying to these failure probabilities can be achieved by selecting components available on the market that contribute to this by having features that reduce the probability of occurrence of certain failure processes, such as a pipe with filter to prevent clogging or adding a heat element in the chamber to prevent frost. Alternatively, components associated to the failure processes should be tested and designed in such a way that they comply to the failure probability requirements. In conclusion, for a potential follow-up design, a detailed fault tree analysis should be performed with scientifically or statistically supported values for the failure probabilities. In this thesis no further elaboration is done on this.

Furthermore, as was mentioned in the previous paragraph, the failure probability for the closing mechanism follows from the failure probability requirement regarding non-closure and the closure demand frequency which is denoted as the probability of inflow. Herein, it is important to realise, that the closure demand frequency is determined with the probability of occurrence of a critical water level resulting from Hydra-NL. Hydra-NL uses the current statistics and prediction models. This means that in for example five years with updated statistics and new prediction models, the closure demand frequency could increase which has in turn an effect on the failure probability requirement of the closing mechanism which in fact is designed for 40 years. As a result, the design then may simply no longer meet the requirements according to new models. This means that a detailed fault tree analysis needs to incorporate a certain margin to compensate for this.

Initially, in the determination of failure probabilities of the system components based on the requirement for non-closure, the drive mechanisms were assumed to be separated from each other, meaning the concrete chamber was compartmentalised and each pipe was the drive mechanism for one concrete chamber. However in the structural design it appeared that a continuous chamber was required to efficiently connect the six gate parts with respect to the water tightness by rubber seals. The result of this is that all pipes are connected to one continuous concrete chamber. They are still independently working from each other, but there is now one main drive mechanism. This contradicts the initial specified fault tree where the failure probability requirement was divided over six gate systems with six drive mechanisms. Conveniently, this change is actually beneficial for the robustness of the barrier since failure of one pipe does not stop the barrier from moving. It only affects the closing time since the total inflow becomes less but still for the same continuous concrete chamber. Furthermore, non-closure because of failure of the drive mechanism, is when the closing time increased to a too long duration where the gate can not close in time. This occurs when a number of pipes fail or are clogged. The probability of occurrence for this is relatively low, since the pipe are still independent. This means that the fault tree obtains more margin in the rest of the failure probabilities for the components.

7 Conclusions and Recommendations

7.1. Conclusions

Based on the objective, the following principal study question was:

- How does a conceptual design of an adaptive self-closing flood barrier for an upper river region in the Netherlands look like?

As an aid to answer this question, a 3D impression of the self-closing flood barrier retaining the design water level in the area is shown in Figure 7.1.

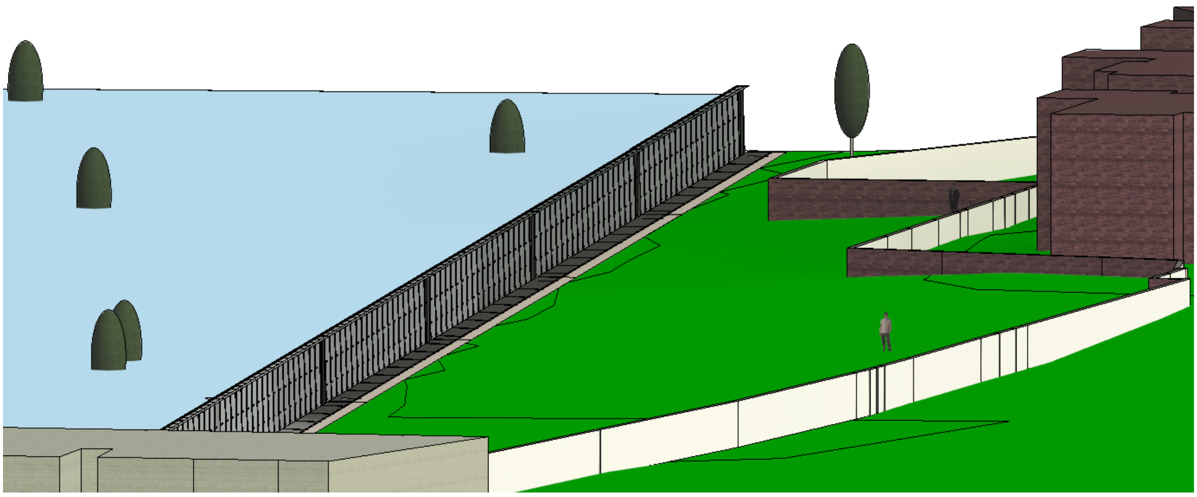


Figure 7.1: 3D impression of the self-closing flood barrier in the project area

7.1.1. General conclusion

The flood barrier is able to, as the name already implies, close itself. The self-closing principle involves a floatation mechanism which is driven by water flow filling a floatation chamber through pipes that are connected to the Meuse river. This means that the system automatically fills with water at a certain water level in the Meuse. The barrier floats in the water filled chamber and moves upwards with a rising water level inside the chamber. The barrier consists of a floater or an air tank which provides sufficient buoyancy and maintains the floating stability while moving upward. Furthermore, the retaining part has a flat type configuration.

After the peak of the high water event, the system drains the water automatically as the water level in the Meuse drops. This leads to re-opening of the barrier, i.e. lowering of the barrier. Multiple independent pipes are responsible for the inflow providing a certain robustness to the system. As a recovery measure the system can be manually filled with the help of a water truck.

The design is developed for an area in the upper river region in the city of Arcen in Limburg along the Meuse over a span of 240 m. This city is characterised by its cultural-historical values and its strong connection to the Meuse. For this reason the city attracts many tourists throughout the year. This makes it difficult to integrate a conventional water barrier such as an earthen dike for example. The area has little available space and asks for an alternative solution, which is why the self-closing flood barrier has high potential for this area. A remarkable characteristic of the barrier's location is that it crosses private gardens of residents living in the close proximity of the Meuse. This location is proved to have the most potential to construct a self-closing flood barrier as such by taking into account the boundary conditions. However, the private gardens occupy a lot of the area and are hard to avoid with construction of such a barrier.

The barrier is able to withstand a retaining water level of 2.44 m locally above the ground surface level. This is a relatively high water head in comparison with other presently applied self-closing flood barriers such as in Spakenburg for example where the retaining water level is merely 1 m above the ground surface level. In respect of future changes, the barrier has an adaptivity feature which allows the barrier to increase in retaining height.

Furthermore, the barrier is executed with rollers to provide lateral guidance. The gate is relatively slender with a maximal width of 100 mm for the most part of the spans and is low in structural utilisation. In rest, the gate is invisible because it is fully submerged into the underground structure improving the integrability. This also means that the structure does not require support structures above ground surface level which would normally prevent obstruction of sight. The floater of the barrier is compartmentalised in order to improve the robustness. The gates are water tight connected with seals that activate when water presses against it, making it self-waterproofing.

There are some new insights which presented itself during the process. Initially it was expected that the flat gate required less excavation but it has been proven to be rather the opposite. The excavation depth results in almost 5 m and the excavation width in almost 4 m. This has consequently strongly an impact to also the concrete thickness. It asks for a heavier substructure to just to meet stability requirements. This automatically drives up the costs as well. Furthermore, with the application of seals throughout the structure and the pipe system, there is more maintenance required in contrast with the initial expectations. On the other hand, the barrier is only in use for extreme water events, for once in five years on the average, which could make this more manageable.

Nevertheless, it has been proven that the structure is a technically verified solution that is applicable for a self-closing flood barrier in the upper river regions facing less available space and restrictions regarding the spatial quality as the solution is characterised by its integrability, adaptibility and simplicity.

Below are some quantitative characteristics resulting from the design:

- 274 tonnes kg steel
- 1524 m^3 concrete
- 2600 m^3 water in system
- six gates with 40 m span each

7.1.2. Answers to the subquestions

In this section, the subquestions will be answered one by one.

How does the length of single structures as part of the self-closing flood barrier relate to the failure probability requirement for a dike section, considering the failure mechanism "height of

water retaining structure" and how does this result in the assessment?

In this project the length-effect for the height of the barrier is N = 1, which in fact means that the failure probability for height for the dike segment is the same as the failure probability for the dike section. The length-effect is determined by the uncertainty in the strength and for the failure mechanism overtopping/overflow, the strength of the structure is translated to the height of the structure, which is in this design is chosen identical over the entire dike segment. The height is measurable and therefore it can be executed precisely as intended, which eliminates the uncertainty. For this reason N = 1, and this results in the same maximum failure probability for the height for each dike section, but also the entire dike segment.

Fluctuation on the load side, meaning the actual hydraulic load level consisting of water level and wave height may still occur, but this is accounted for in the height of the barrier. For this reason the top height of the gate is at NAP + 18.23 m, whereas the water level is NAP + 17.84 m and the significant wave height is 0.25 m. So in other words, the calculated retaining height, will be sufficient for every meter span of the dike segment to resist the hydraulic load level and the associated overtopping discharge with a failure probability of $\frac{1}{417}$. This is because of the fact that the barrier has a known identical height over the span.

How does the length effect relate to the failure probability requirement for a dike section, considering the failure mechanism "reliable closure" and how does this result in the assessment?

The length-effect is determined by the uncertainty in the strength, which in this case is the reliability of closure. The uncertainty in the reliability of closure follows from the design. The design choice that plays a role in this for the failure mechanism non-closure is the number of closing hydraulic structures within the dike segment that can contribute to non-closure. Thus, when a design is made where the barrier consists of more individual parts but with shorter spans dividing the entire barrier over more dike sections, i.e. into more independently closing parts, the more stricter the failure probability requirement will be to which is designed for. On the one hand, one would say that it may be more difficult, but on the other hand, the length of the dike segment is fixed, so dividing into more dike section, means smaller dike sections, meaning in turn that per dike section failure events such as for example an obstacle on the barrier have less probability of occurrence for one dike section. Thus, designing with stricter requirements may in that case not be as difficult as it seems beforehand.

So the length-effect for the failure mechanism non-closure is a result of the design choices. For example, one could choose a design for a self-closing flood barrier with more shorter spans leading to inflow pipes with a small diameter and a reduced risk of clogging by large objects. The chosen subdivision of the segment into a number of closing self-closing flood barriers, follows in what is the length-effect. From this, the associated probability of failure per dike section follows in turn, to which the closing processes and components are further designed to comply to the requirements related the closure of the structure.

From the perspective of the loads, the larger the dike sections, the more inflow is possible in case of failure and a following flood. This has an effect on the critical flow velocity and inundation rate, which determines at which water level in the Meuse the structure needs to be fully closed. The probability of this water level is thus the probability of inflow or the frequency of closure demands per year. In this design project however, the probability of the water level in the Meuse for closure does not depend on the the width of the gap in the dike section in case of a failed to close barrier. The reason for this is because any flow through an opening is not allowed, since erosion protection is not present or deliberately not applied for spatial quality reasons. In this project, the probability of inflow or the frequency of closing the barrier is determined by the probability of occurrence that the water level in the Meuse reaches the sill height of the structure which is the ground surface level at the location of the structure (NAP + 15.4m).

The dike segment in Arcen is subdivided in five dike sections. This is based on site-specific characteristics and boundary conditions. For each dike section the part of the self-closing flood barrier has its own uniformity. This means that it is considered that each dike section has an individually closing barrier connected to the other dike sections. Each barrier functions independently of the others. Thus there are five closing hydraulic structures and the dike segment also consists of an already present pumping station that is considered as closing hydraulic structures. This determines that the length-effect for the failure mechanism non-closure is $N = 6$. Each dike section has its own closing barrier with its own uncertainty in the reliability of closure. Thus taking this input into account it results in the division of the failure probability requirement for non closure to $6.7 \cdot 10^{-5}$ per dike section. With this subdivision and the associating failure probability requirement derived from the standard, the assessment for dike section 2 resulted with the conclusion that designing on the probability of non closure is proven feasible. The reason for this is that the order of magnitude of the individual failure probabilities of components and failure events are realistic and in line with practice. However, for a potential follow-up design, a detailed fault tree analysis should be performed with scientifically or statistically supported values for the failure probabilities.

Within each dike section there is also a subdivision in the number of closing gates comprising the entire barrier for one dike section. This is denoted with n_{gates}. The more (smaller) gates, the higher the probability is that one gate fails but with a slower overall inundation than with a design with less (longer) gates. In both cases erosion will occur with structural failure as ultimate outcome. Structurally it does not make much of a difference as well, because the floater and the gates are compartmentalised anyways and fully "clamped" to concrete in retained position. The rate of inundating the chamber on the other hand is affected by the number of gates within one section. However, this is considered not significant with respect to other aspects.

Which driving mechanisms (i.c.w. retaining wall) are suitable solutions that are in line with the failure probability requirement for a dike section for the reliability closure assessment of the structure?

The flap and flat gate were the most potential gate types after evaluating solely between barrier types. The main criteria on which these barrier types had a clear advantage over the others were the adaptibility of the gate and the integrability in the area. The flat gate was evaluated as the best one of the two.

After including the possible drive mechanisms in the selection process, it became clear that buoyancy was the most reliable option. The main reason that a buoyancy driven system comes out of the evaluation better is because it does not require machine rooms that occupy additional space and require more installation works. For a buoyancy driven system, the operation is easier and does not have to rely on mechanical processes. Only water is need which is already plenty available, particularly in the project area considered since the structure is located along the Meuse. Also the drive system has sufficient back-up, because if one pipe fails there are other pipes still working that fill the system, because the concrete chamber is one continued basin over the entire span. In such a case, the duration of gate closure will admittedly increase, but that is not as much of a problem of an entire section of the barrier not being able to close. Failure of the entire drive mechanism would imply that a significant number of pipes fail to work, leading to a three or six times longer duration, which will increase the risk of flooding. This has however a small probability, because the pipes are part of a parallel functioning system.

The other drive mechanisms come with more mechanical parts and a power supply that also require more maintenance. A lack of maintenance can lead to failure. Besides, more mechanical parts or subprocesses also means more interdependencies, which is not beneficial to the failure probabilities considering the fault tree. It makes designing on probability of non-closure harder. However, if the associated parts are designed in such a way that they comply to the failure probability requirements, it is still feasible. But generally it is preferable to keep the design as simple as possible. Furthermore with a floating mechanism, there is always an option to manually fill the system with the help of a water truck that is being called, which still makes it a system that does not rely on mechanical parts because only water needs to be filled.

How can adaptivity be incorporated into the design for the retaining structure and the retaining design height, taking into account a reduced design lifetime for the retaining function? (note:

this does not include the foundation)

Adaptivity is taken into account by ensuring that the embedded structure has a reserved height remaining inside for heightening of the gate with additional material to comply to new hydraulic boundary conditions. The determination of the required free height is based on the boundary conditions for the most extreme, unfavourable situation, where thus KNMI climate scenario 'W+' is applied for a functional design lifetime of 100 years. The actual design, which is adaptable, may be designed with conditions associated to a milder climate scenario, such as for example 'G', for also a functional design lifetime of 100 years. This is what is recommended in the Design Guide of Hydraulic structures. However, the goal is to design with a shorter functional lifetime in order to be more responsive to current statistics and prediction models and leave room for adaptivity to the design for the assessment period after the functional lifetime. In order to comply to both the Design Guide as to the stated goal, hydraulic conditions were sought with climate scenario 'W+' but with a shorter lifetime that would be similar to conditions as if were to be designed with climate scenario 'G' for a functional design lifetime of 100 years. This was the case for a functional design lifetime of 40 years and thus with climate scenario W+.

The associated retaining height for this case is 2.83 m. The associated retaining height with a maximally adapted design is 3.24m. This means that the design of the embedded structure has a remaining free height inside of 0.41 m for adapting the retaining height of the original gate. Taking into account adaptibility in the design seems in the ends valuable, because the considered gate heightening possibility of 14% is substantial. However, a cost-analysis has not been implemented to examine whether it is also cost-effective. The only difference with the design as of yet is that the gate has less height, which saves steel. The adaptation itself involves simply attaching more material with a similar shape to the top of gate. The structure is designed with sufficient margin in the structural utilisation of the elements. Additional height which might be required in the future may add more weight to the structure but the floatability is also still ensured.

What does the integration and connection of different dike sections within a dike segment look like with minimal deterioration on spatial quality and an aim for zero houses being excluded from protected area by the dike (in Dutch: buitendijks)?

Integrating the self-closing flood barrier in the project area with minimal deterioration on spatial quality means a fully submerged structure in the ground without townscape obstructing support structures above the ground surface. The structure is completely invisible at the times when it is not in use, which is necessary because the structure is located in private gardens. Locating the structure in private gardens is an effect of preventing houses being excluded from the protected area. When the barrier is closed, it structurally means that the moments resulting from the hydraulic loads are fully taken by the concrete support structure. This leads to a heavier concrete structure. Although the structure minimally affects the spatial quality when constructed, during construction this is almost unavoidable.

With respect to the connection of barriers withing the dike section, the barriers are connected with a seal ensuring water tightness. The seal is automatically activated when water pressure is exerted on it. The barriers are not structurally connected as to ensure the independently closing property which is important for the reliability of closure. The concrete chamber in which the barrier stays afloat is continuous over one dike section. Between dike sections there is a separating wall that does extend above ground level. However, these are easily integrable within the existing retaining wall or other present structures in the area and not affect the spatial quality of the area, as these are also not located withing private gardens. This is mainly due to the predetermined boundaries of the dike sections for the self-closing flood barrier. Other dike sections that are not considered in this thesis also have the option to be connected to existing structures to which an end support can be constructed or integrated into earthen dikes.

How does the design process change, especially considering the hydraulic loads, if the design should also be made for a location with a different failure probability requirement and boundary conditions, and then what are the final changes in the design?

If for a location in Dordrecht at quays along for example the rivers Oude Maas or Beneden Merwede a movable hydraulic structure needed to be constructed that integrates in the area where there is less space or not suitable to build an earthen dike, a self-closing flood barrier would be an option to consider following the same design process as in this report. However, the design would most likely majorly differ from the design in this thesis as was briefly outlined. For example the solution would be more similar to the self-closing flood barrier from Hyflo Bv in Spakenburg, but maybe on a larger scale. Another option would be a totally other gate concept such as an air filled bellows barrier or a flap gate such as the to be build Vlotterkering in Steyl-Maashoek.

For the case with Zeeland, a structure as presented in this report will not be suitable. The expectation is that, when considering a similar structure, the boundary conditions in Zeeland will not result in the same sized self-closing flood barrier as in Arcen. For example it could be used within city-centres but on a smaller scale. On the other hand, it may be a solution to integrate within earthen dikes as an alternative for dike heightening, if for example dike heightening contravenes with spatial quality or townscape just as with Arcen. In this configuration, the structure directly functions as a dike reinforcement within the dike as well. Very important with considering such a solution direction is the cost-efficiency of the structure.

7.2. Recommendations

Statistical analysis and research on individual failure probabilities
In order to verify the fault tree analysis regarding the closure process to check whether there is compliance with the reliability requirements, it is advised to do a statistical analysis or research on failure probabilities of individual elements or events associated with the design. This will give insight if a design is possible or other measures are required in order to comply to the reliability requirements.

Costs
In the selection process for a gate type and driving mechanism, costs are not included in this design study. However, it might have a large impact on the value of particular concepts, leading perhaps to another outcome, provided the cost estimate is appropriately done. With this also an ECI (Environmental Cost Indicator) should be determined.

Make a structural design for temporary structures in construction phase
In this project the focus lies on the failure mechanisms for the permanent structure which is structurally designed for. The failure mechanisms for the temporary structures in the construction phase are outlined but not elaborated with a structural design. For a complete design, the structural design also extends to the construction phase. The structural design for the failure mechanisms in the construction phase results in for example which type for a sheetpile wall should be chosen and what the embedded depth for this sheetpile wall should be. Furthermore, sheetpile walls need anchoring or struts to keep them in place during the construction. These checks are important in order to ensure a safe construction environment, but will not influence the permanent design. However, these checks are part of the temporary structures and are therefore omitted in this thesis but they are recommended for a complete design.

For uplift the governing situation is in the use phase for the permanent structure but it is also checked for the the underwater concrete floor in the construction phase. However, this additional check is not reported in this thesis.

Concrete design with steel reinforcement
This design study focused mainly on the steel barrier, the functionality and the stability leading to initial dimensions and shape for the concrete structure. However it is still necessary to check whether the concrete compressive strength is not exceeded and how much steel reinforcement needs to be applied to take up the tensile (and compressive) and shear forces. The loads on the concrete walls resulting from the hydraulic load have already been determined. The walls are plated elements and should be modelled as such in for example a finite element software such as DIANA. The walls are additionally subjected in governing situation by soil and groundwater pressure from outside the chamber and hy-

drostatic water pressure from inside the chamber. The support of the walls can be modelled as fixed supports or rotational springs, depending on the rotational stiffness of connection of the wall with the floor. This depends on the amount of reinforcement that is applied. The governing wall in this case is perhaps the wall on the river side, which is subjected to inner hydrostatic pressure and from outside an active soil pressure which is not as large. The nett loads on the wall on the right hand side cancel each other significantly out.

Another governing situation that must be checked is the situation where the concrete chamber is empty but the groundwater has risen significantly which exerts a load on the concrete wall. The governing wall is the wall on the dry land side which is subjected to passive soil pressure which is governing.

From the governing situation and reaction forces the floor can be designed on concrete strength and steel reinforcement. The floor can be modelled with either a support as a bedding or on two simple supports. Both results should be compared. The floor is subjected to the reaction loads of the walls and the nett vertical water pressure.

Hydraulic pipe design for inflow
The inflow through the pipe system majorly determines the filling duration and thus also the closing time of the barrier. The inflow through the pipe system depends in turn on the application height, which determines the head and the diameter of the pipe. It is recommended to specify this in a follow-up design. The height of the application of the pipe determines actually the second governing situation that holds for uplift and the concrete strength. The application height of the inflow pipe determines to which level there is no inflow in the system and the groundwater is allowed to rise. A risen groundwater table with simultaneously an empty concrete chamber is the governing situation for uplift and concrete strength.

The hydraulic pipe design mainly consists of using Bernoulli's principle. Herein the water head, the pipe diameter and filling duration can be adjusted until a desired solution is found, where the result is a level of application of the pipes and a diameter.

Also important for the hydraulic pipe design is to include the locations of the pipes in horizontal plane. Placing the pipes need to take into account already present cable and pipe works (Dutch: KLIC).

Detailed gate design
For a complete design it is recommended to increase the level of detail for the gate, meaning that the steel gate design should be optimised, since some components are very low in structural utilisation. There are still opportunities to design more efficiently to save steel. For example the amount of columns can be reduced, resulting in longer subspans. the loads on the columns will increase which should be closely monitored, since the columns were the most governing components of the barrier with respect to the structural utilisation.

Furthermore, the seals can be designed, which involves choosing a thickness and the material. Also, the rollers can be designed from which additionally a friction force follows that in turn can be taken into account for the gate design as an extra load. Lastly, the consoles to support the cover deck and the cover deck itself can be designed to find a suitable profile and respectively a thickness.

Validation of the final design
The final design of the self-closing flood barrier has not been validated in this project. It means that the societal reception is not surveyed by conducting interviews with the stakeholders, such as the residents and the Water Authority in Limburg. This is recommended as it gives insight in whether this design is actually feasible to construct in terms of acceptance by the stakeholders. The design could possibly need adjustments as a result of this validation or may not be accepted at all.

Appendices

A

Current applications of self-closing flood barriers

This appendix presents the currently available solutions on the market for applying a self-closing flood barrier.

A.1. Hyflo BV Self-Closing Flood Barrier

Hyflo BV developed a self-closing flood barrier where the flood wall floats and subsides inside a basin that fills up with water in case of a flood, see Figure A.1.

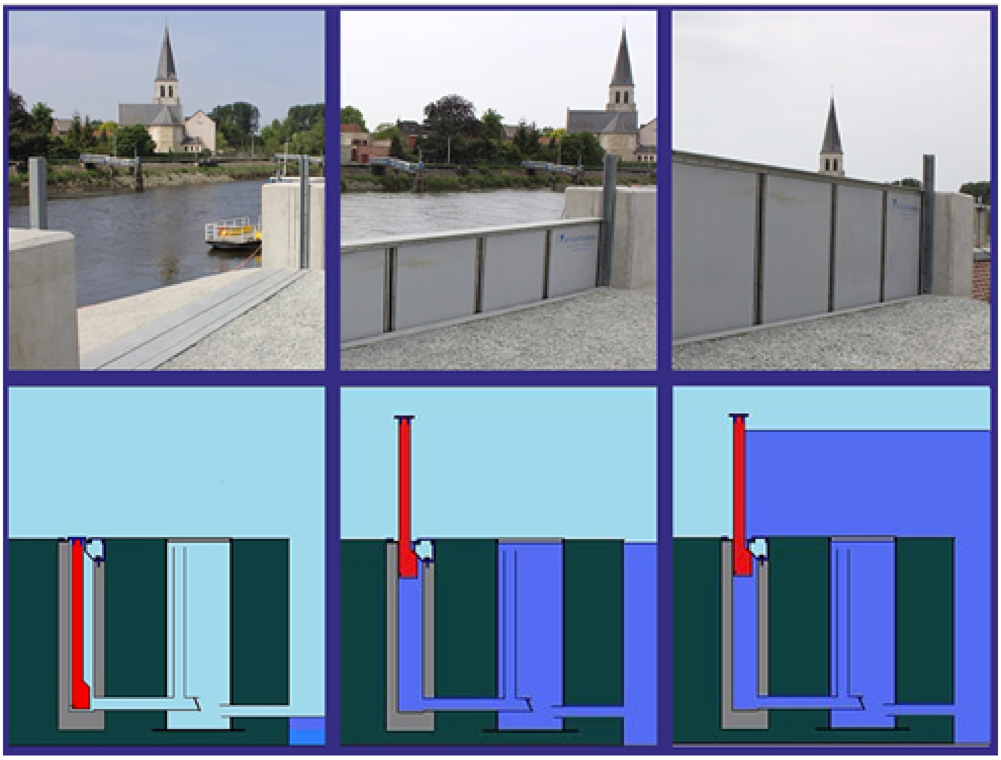


Figure A.1: Schematised view of the mechanism of a self-closing flood barrier (Hyflo BV, 2021)

To provide an impression of such a self-closing flood barrier, Figure A.2 shows a table from Hyflo BV indicating the dimensions in which the already existing Self-Closing Flood Barrier^{TM} can be equipped with an implemented steel basin and a maximum pressure capacity of 400 kN for urban and rural areas (Hyflo BV, 2021). This is one of the types they offer amongst others. For higher loads, the structure could be executed with a concrete basin.

	Protection Height	Max length	Height	Top width	Bottom Width	Pipe connection
SCFB 500 S	500 mm	8000 mm	1065 mm	490 mm	700 mm	160 mm
SCFB 1000 S	1000 mm	8000 mm	1645 mm	490 mm	800 mm	160 mm
SCFB 1250 S	1250 mm	8000 mm	1945 mm	490 mm	850 mm	160 mm
SCFB 1500 S	1500 mm	8000 mm	2245 mm	490 mm	900 mm	160 mm

Figure A.2: Types of executing Self-Closing Flood Barrier$^{\text{TM}}$ (Hyflo BV, 2021)

These involve mostly applications at breaks and gaps within a dike or levee section (in Dutch: coupures). The loads on these barriers are transferred to supporting structures above surface level or to the connecting structures with the dike section or levee section. One application by the Belgium firm Aggeres was done in a port in Spakenburg that concerned a protection length of 330 m (Aggeres, 2017). This is currently the largest contiguous protection length for a self-closing flood barrier. In Table A.1 some specific characteristics are mentioned.

Specific characteristics	**Conclusion**
slender retaining structure	little space consumption
use of bearings on both sides above surface level	obstruction of view
large foundation depth for large retaining height	costly and laborious foundation
large foundation width for basins	costly and laborious foundation

Table A.1: Overview of specific characteristics of Self-Closing Flood Barrier

A.2. Vlotterkering

Another solution is the ’Vlotterkering’ developed by Vlotterkering BV. This solution involves a hinged system with a steel panel and one or more floaters underneath in a concrete basin. In case of a flood, the basin fills up with water and the floater starts floating allowing the steel panel to rise to its retaining position, see Figure A.3. When the barrier is not retaining, it’s concealed into a flood protection or the ground as to ensure no obstruction of the view. (Vlotterkering BV, 2023)

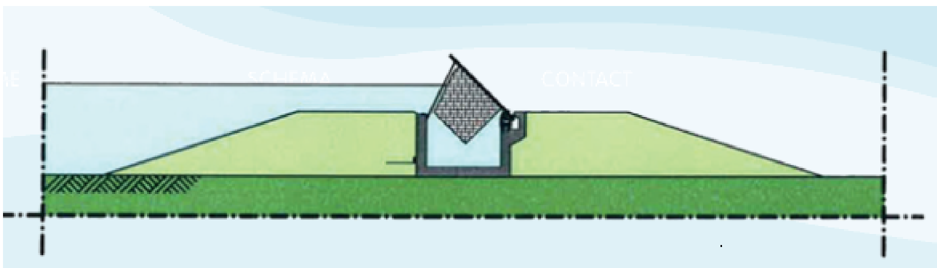


Figure A.3: Schematic view of ’Vlotterkering’ in retaining position (Vlotterkering BV, 2023)

The ’Vlotterkering’ is planned to be constructed in Steyl-Maashoek in Limburg as part of the Flood Protection Program. The length of the segment is 200 m and has no intermediate supports above ground level (Waterschap Limburg, 2023). The loads are fully taken by the structure and the foundation.

Specific characteristics	Conclusion
large foundation width for large retaining height, because of hinged retaining wall	costly and laborious foundation
not a slender structure, because of attached floaters	large amount of space intake when active and large foundation depth
concrete basin	costly and laborious foundation
no bearings on the sides	no obstruction of view when inactive
larger spans, larger amount of water necessary	long deployment time or extensive intake structure
retaining height independent of foundation depth	adaptible retaining height

Table A.2: Overview of specific characteristics of Vlotterkering

In Table A.2 some specific characteristics are mentioned.

A.3. Aggeres ’kleppenkering’

Aggeres developed a gated barrier which is activated by a water sensor or manually in case of a flood. When activated, pneumatic pumps start working and raise the barrier. The maximum span for one single retaining element is 20m and the retaining height can reach up to 1.8m, see Figure A.4 (Aggeres BV, 2022).

Figure A.4: Picture of ’Kleppenkering’ (Aggeres BV, 2022)

In Table A.3 some specific characteristics are mentioned.

Specific characteristics	**Conclusion**
small foundation depth	low-cost and simple foundation
slender retaining structure	little space intake when active
large foundation width, with large retaining height	large amount of space intake when inactive
no bearings necessary on the sides	no obstruction of view
small spans available, for a large span many individual elements required	extensive connection design and higher risk to leakage

Table A.3: Overview of specific characteristics of ’kleppenkering’

B
Methodology

B.1. Methodology: standard civil engineering design cycle

Because this thesis involves a design topic, the standard civil engineering design cycle will be used in each design loop of the project. This is illustrated in Figure B.1. Each design phase will consist of these design steps.

Analysis
Every design cycle begins with an analysis. This consists of for example a process and functional analysis which eventually establishes the requirements, for instance functional requirements or structural requirements. In addition, the boundary conditions and how they may change in time and the evaluation criteria are also documented. From this design step, follows which components are needed for the design and with each design loop, the analysis step leads to an understanding of how these should be further detailed in the next design step.

Synthesis
This design step will explore possible variants to meet the requirements, either functional or structural. The intent is to visually identify which directions for the solutions are possible. For example, the necessary components should be determined, the available options for these should be explored and in what way these will cohere. Or for instance, in a further design step, there should be chosen what type of cross-sections will be used for the structural elements and how these choices will effect the load distribution and load transfer to the soil. Ultimately this will be followed by (global) dimensioning of the elements and creating a design by combining the components. In the end of this step, each design loop will have a visual representation of the design or design alternatives with the level of detail associated with the design loop.

Verification/Simulation
The simulation or verification step considers whether the design of the alternatives created can be built effectively or realistically, whether the functionality is in line with the functional requirements and whether it makes sense beforehand in terms of, for example, force transfer or dimensions. The designs or design alternatives in their level of detail will be assessed relative to the requirements of the associated design loop. If some requirements could not be quantified, a verification should be done by assessing the effect of a concept, if necessary with the help of an expert using engineering judgement from experience.

Evaluation
In this design step design alternatives are valued based on multiple evaluation criteria, followed by a trade-off matrix in the spatial-functional design loop or for instance by evaluating the final design to multiple criteria such as feasibility, construction time and costs, aesthetics, environmental footprint, social safety and maintenance the after the structural design phase. Ultimately at the end of this step, in each design loop, a decision is made for the final design or final design alternatives for that particular

design loop which will be further detailed in the next design loop.

Integration
Subsystems should eventually be integrated into one complete functioning system. This step will have clear focus on interface requirements on a lower detail level and on spatial integration on a higher level of detail (Voorendt, Molenaar, 2022).

Validation
In the last design step the final design is validated to check if the design objective is accomplished. Also it is checked if the requirements are applicable to the design and if the design method is suitable (Voorendt, Molenaar, 2022).

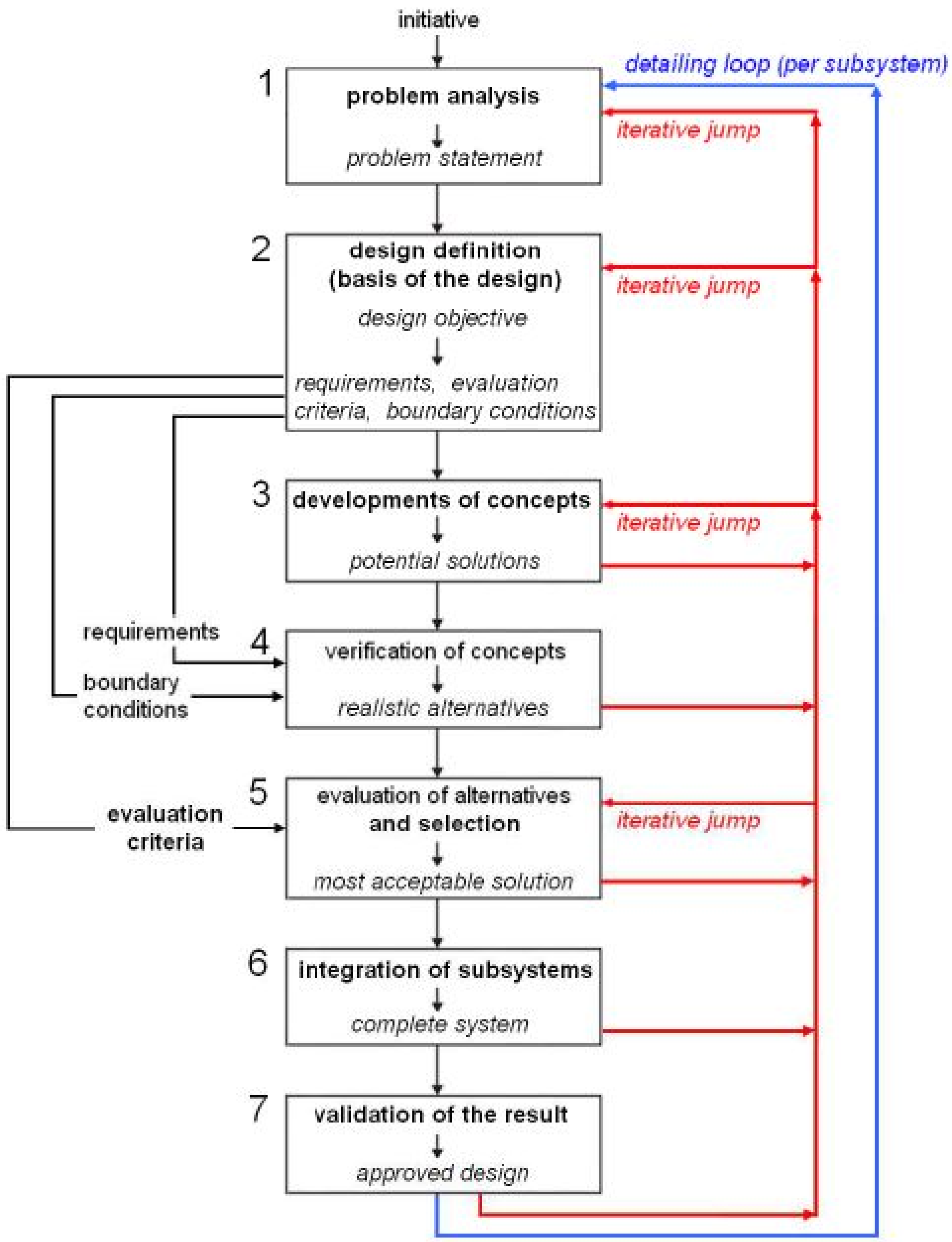


Figure B.1: The elementary design cycle in civil engineering (Voorendt, Molenaar, 2022)

B.2. Application of methodology

System analysis

Site analysis
The second part of this section contains the description of the project location that will be used as an example for which the self-closing flood barrier will be designed. The case concerns a location in Arcen, Limburg for which a dike reinforcement task is ongoing to reinforce a dike segment of 5.1 km, see Figure F.1.2.

Functional analysis
The functional analysis is an examination that will provide a functional overview of the self-closing flood barrier. This could be presented in a tree structure in which the functions, subfunctions and their relations are shown (Voorendt, Molenaar, 2022).

Stakeholder analysis
The stakeholder analysis is aimed at finding all parties directly or indirectly involved and affected by the problem and the potential solutions. In this analysis, also the specific objectives, responsibilities, contributions and importance to the project are defined that lead to requirements and evaluation criteria for the design of the structure. The weights of the criteria with respect to each other could be determined by creating a stakeholder matrix as a tool to review influence and involvement of each stakeholder and thus also their wishes and requirements.

Basis of design
Prior to the design loops, the basic principles, the functional requirements, boundary conditions and evaluation criteria that the self-closing flood barrier should have, should be determined. Main functions of the system that are clearly explained in the functional analysis in section B.2 lead to the functional requirements. Hydraulic boundary conditions will be derived from Hydra-NL.

Hydra-NL is an application using a probabilistic model that provides the hydraulic loads, such as water levels including uncertainties and is consistent with the BOI (in Dutch: Beoordelings- en Ontwerpinstrumentarium), for all primary dikes and engineering structures in the Netherlands to assess their safety.

Design loop 1: Spatial-Functional Design

Selection of project location
In this first step of the spatial-functional design, the exact project location of the structure is selected based on predefined location requirements and evaluation criteria. These requirements and criteria follow from the site analysis and the boundary conditions.

Preliminary selection of gate type
In this subsection will be covered which possible gate types are available for a self-closing flood barrier. They will be verified against the list of functional requirements and evaluated further with respect to criteria in a Multi-Criteria Analysis.

Final selection of gate type in combination with drive mechanism
This phase starts with developing concepts in which possible solutions for the drive and closing mechanism will be discussed and visualised. The concepts are then in second selection procedure step evaluated on several criteria in order to determine the which concept has the highest potential. This section will answer the following deepening question:

> *”Which driving mechanisms (i.c.w. retaining wall) are suitable solutions that are in line with the failure probability requirement for a dike section for the reliability closure assessment of the structure?”*

The evaluation will be done by multi-criteria analyses (MCA) to assign measurable values to the alternatives in terms of for example material use, robustness, sustainability, adaptability, reliability and integrability, maintainability and efficiency. The high potential concept is continued with in the next design loop.

Finalisation spatial-functional design
Lastly, the concept will be designed with respect to its functionality. Important components will be elaborated on and global dimensions will be determined. Also, in this section attention will be given to the adaptibility, reliability and integrability of the structure. Furthermore, this section will elaborate on the connections between contiguous single elements of the dike sections where attention is also given to water tightness. This step will partially give an answer to the deepening questions:

> *"How can adaptive building be incorporated into the design for the retaining structure and the retaining design height, taking into account a reduced design lifetime for the retaining function?*
>
> *"What does the integration and connection of different dike sections within a dike segment look like with minimal impact on spatial quality and an aim for zero houses being excluded from the protected area by the dike (in Dutch: buitendijks)?"*

More information on dimensioning with the respect to the retaining height and the reliability of the structure with respect to the closing mechanism is elaborated below.

1. Height water retaining structure:

Firstly, a verification is done which is associated with the failure mechanism 'height water retaining structure' in the Design Guide for Hydraulic Structures (in Dutch: WOWK). This involves mainly the failure mechanism of overflow and/or overtopping and will eventually provide a critical discharge that is maximally allowed for the structure, based on either the erosion capacity of the soil or the bottom protection or the inundation capacity of the hinterland. The critical discharge is input in Hydra NL together with the failure probability requirement. From this a hydraulic load level will be calculated that corresponds to the height of the structure. This design process will answer the deepening question:

> *"How does the length effect relate to the failure probability requirement for a dike section, considering the failure mechanism "height of water retaining structure" and how does this result in the assessment?"*

2. Reliable closure water retaining structure:

For the selected concept, the design will be further elaborated by determining the failure probabilities of the critical components and processes in the closing mechanism. Herein, the division of the dike segment into dike sections is important and determines the length-effect. This addresses the aforementioned second failure mechanism of the Design Guide for Hydraulic Structures (in Dutch: WOWK) and provides an answer to the second deepening question in this thesis:

> *"How does the length effect relate to the failure probability requirement for a dike section, considering the failure mechanism "reliable closure" and how does this result in the assessment?"*

Design loop 2: Structural Design
The second design loop of the thesis will consist of the structural design of the high-potential alternative. The structural design involves a semi-probabilistic method to verify the flood barrier to the structural requirements of the Dutch Water Act and the Building Code including the Eurocodes. This addresses the last failure mechanism of the Design Guide for Hydraulic Structures (in Dutch: WOWK). In this design loop, special attention is given to the load transfer to the foundation and the soil, since one of the design challenges is to omitt the use of support structures above ground level to prevent obstruction of the view on the city or village. Also, the failure mechanism 'piping' is part of the structural design for which the assessment extends to only the "detailed test" which is performed completely deterministically. The structural design phase will be subdivided into four sub-phases.

Constructibility
In this section, it is essential to identify construction method, the excavation technique, the foundation method, transport and logistics from and to the construction area and finally the construction sequence. Also, the failure mechanisms are identified, which follow from the construction sequence. The govern-

ing situations for the failure mechanisms will determine the load situations and load combinations.

Stability
The interaction between soil and structure will be verified in this section, which encounters the vertical, horizontal and rotational stability and also uplift. The loads acting on the structure and potentially causing instability per failure mode will be identified and will be schematised per load combination. For missing parameters of the structural elements, initial assumptions will be made. If the stability is not yet ensured, dimensions of the structural elements should be adjusted until the unity checks are satisfied. The verification to piping will be done in this section as well. It will follow the schematisation manual for piping of Rijkswaterstaat. The calculation will have a deterministic approach.

Gate design on strength
The geometry is modelled with initial cross-sectional properties. The failure mechanisms for strength are outlined, from which the load schemes are visualised. Each component within the barrier is designed on strength following the Eurocode. If the cross-sectional resistance is not yet ensured, the properties of the structural elements should be adjusted until the unity checks are satisfied.

Generalisation In the closing phase, the design and the design process will be generalised considering what adjustments would be required if the location of the structure or the flood protection system in which the structure is integrated, were to be changed. For instance, other hydraulic boundary conditions and other failure probability requirements will have to be considered. An example is addressing the necessary changes to the structure or the design process if the flood barrier were to be placed in larger cities than Arcen, with higher economical value or more densely populated areas, where spatial integration would require even more special attention. This step will eventually provide an answer to the last deepening question:

> *"How does the design process change, especially considering the hydraulic loads, if the design should also be made for a location with a different failure probability requirement and then what are the final changes in the design?"*

C

Stakeholders analysis

This analysis of the stakeholders of the project contributes to the requirements and criteria to which potential solutions for a self-closing flood barrier must comply to respectively are evaluated upon.

Public Service Providers

- Ministry of Infrastructure and Waterworks
  Dike improvements are executed as part of a national program which the Ministry of Infrastructure and Waterworks is responsible for. The ministry formulates the standards to which dike improvements should be conform to.
- Rijkswaterstaat
  Rijkswaterstaat is a national service provider that works as the executive department for the Ministry of Infrastructure and Waterworks. Rijkswaterstaat is also part of the alliance to work on dike improvement projects for the Flood Protection Program (HWBP). Rijkswaterstaat is also involved as a permit grantor, river management body for the Meuse and an advisor or assessment authority for the dike reinforcement plans.
- Provincial Executive
  The finalised project plan is sent to this governing body which is the authorised body for approval. As the name already indicates, this governing body acts at the level of the provinces. After approval, the realisation phase begins.
- Water Authority Limburg (Waterschap Limburg)
  The project initiator is the Water Authority Limburg since the project involves dike improvements in the area of Arcen which is located in Limburg. The Water Authority is a regional governing body responsible for the management of surface water in the environment. The water authorities are also part of the aforementioned alliance of the Flood Protection Program

Private Service Providers

- Investor
  The investor is the alliance for the Flood Protection Program, which consist of 21 water authorities in the Netherlands and Rijkswaterstaat. The national government finances 50% of the account for the program, where the collective water authorities finance 40% and the individual water authorities finance the remaining 10%.
- Contractor
  ownership of the project, responsible for construction, design. Their goal is to construct the structure safely and in a way that they can make profit.

- Suppliers
  accessibility to bring materials and machinery to project location, supply materials and machines

- Project management team
  they are responsible for the construction process and time and communication between all parties. Their goal is to deliver the project by the planned date and time with minimised delay.

Core stakeholders

- Committee Environmental Impact Assessment
  ecology, flora and fauna, nature

- Spatial quality team
  habitability, integration, townscape, accessibility of Meuse and sight on Meuse

Periphery stakeholders
This group of stakeholders is actually the group for whom the project is meant to be be built.

- Foundation for advocacy for residents along the river Meuse Arcen
  in particular advocating for the benefits for the residents along the Meuse, such as townscape, connection with Meuse, minimum disturbance during construction, quality of their gardens

- Other residents
  townscape, connection with the Meuse, sight on the Meuse, minimum disturbance during construction

- Business owners or farmers
  property loss, being placed beyond the primary flood defence, value of ownership of properties

- Community council
  townscape, connection with the Meuse, sight on the Meuse

D

Boundary conditions and scoring methods

D.1. Surface elevation profiles

In this appendix several cross sections are shown for the surface elevation profiles along the project location.

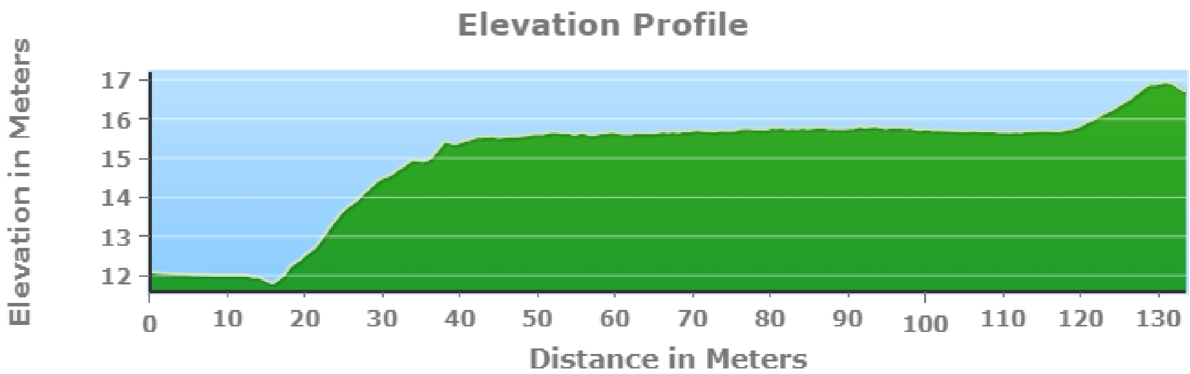


Figure D.1: Cross-section 1

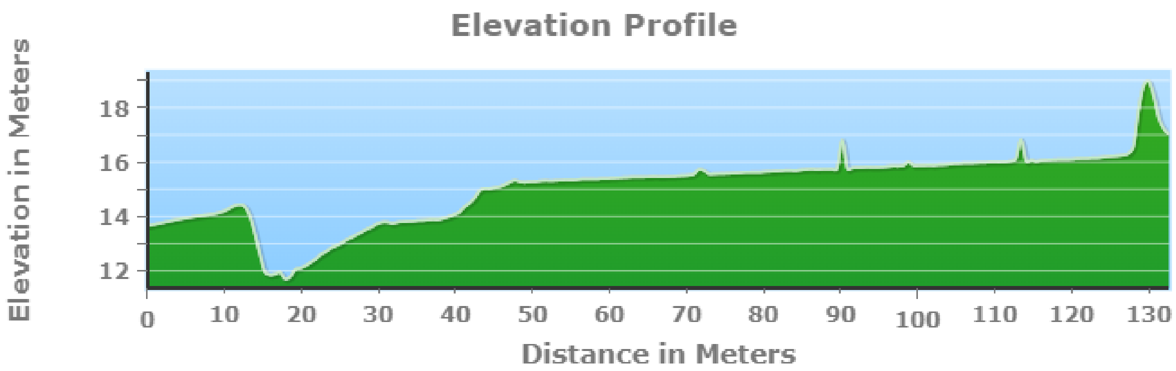


Figure D.2: Cross-section 2

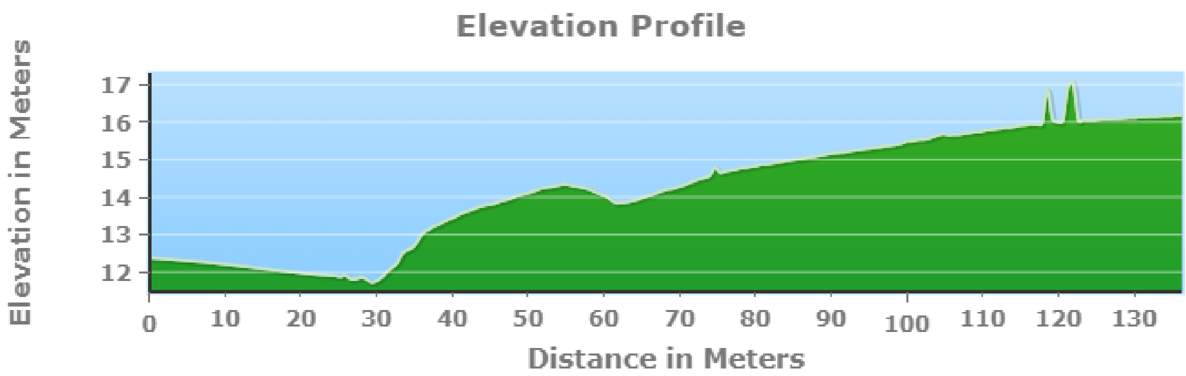


Figure D.3: Cross-section 3

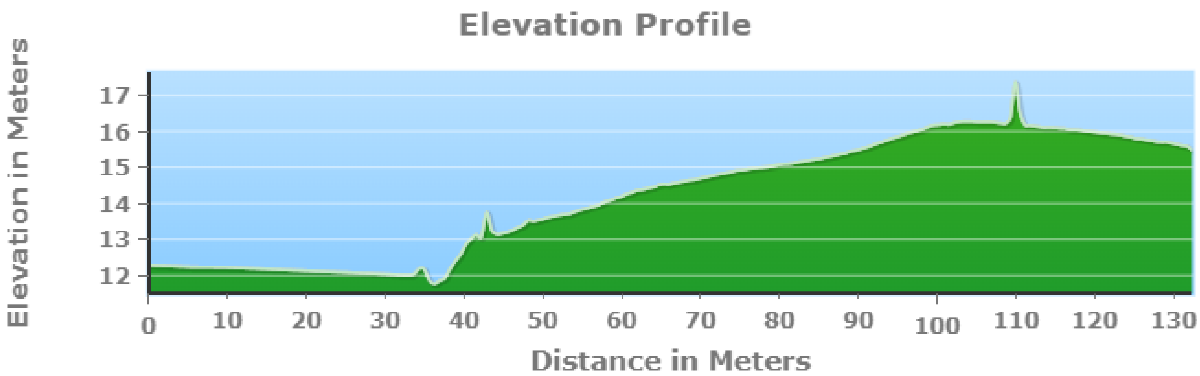


Figure D.4: Cross-section 4

D.2. Soil data

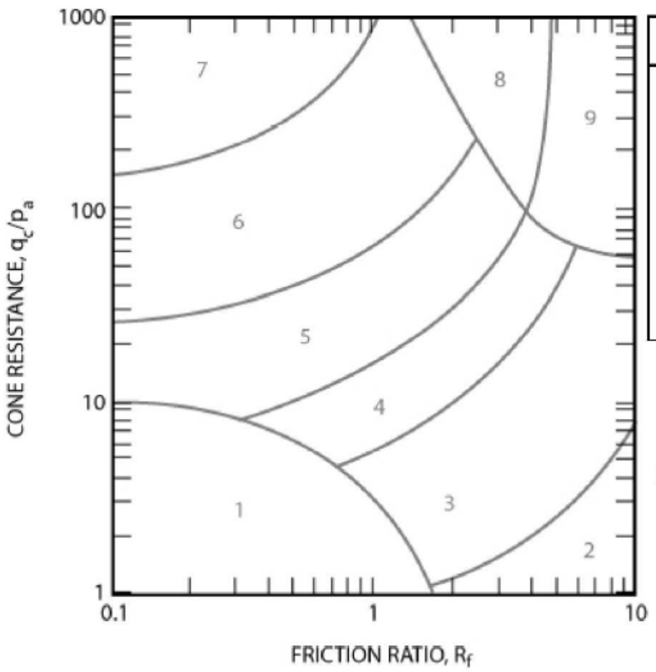
In this section, soil data is presented for locations along the foreland of the Meuse in Arcen-Centre. The central gateway for data and information regarding the Dutch subsoil gives data for only four locations that are relevant for the design, which is shown in Figure D.5

Figure D.5: Locations of retrieved soil data (DINOloket, 2023)

Each location will be addressed below with a CPT-graph and a reconstructed soil profile to the depth of the construction level. The groundwater table is at 12.0 m + NAP which concides with the average water level in the Meuse at that location. The soil profiles are derived from the CPT-graph, the friction ratios and Table 25-3 and Figure 25-1 from the Manual Hydraulic Structures (M. Voorendt, 2023) which are presented below. The specific weights γ and internal friction angles ϕ are obtained from Table 25-6 from the Manual Hydraulic Structures (M. Voorendt, 2023), which is presented in Figure D.8.

soil type	cone resistance [MPa]	friction ratio [-]
gravel	> 4 - 10	0,2 - 0,5
coarse sand	> 2 - 10	0,4 - 0,6
fine sand	> 1 - 10	0,9 - 1,0
silty sand	1,5 - 4	0,8 - 1,4
clayey sand	1 - 2	1,0 - 2,0
loam	1 - 3	2,0 - 4,0
clay, stiff	2 - 4	2,0 - 4,0
clay, medium	1 - 2	3,0 - 5,0
clay, soft	0,5 - 1,0	4,0 - 6,0
clayey peat	0,1 - 0,5	5,0 - 8,0
peat	0,1 - 2,0	5,0 - 10,0

Figure D.6: Indicative values of cone resistance and friction ratio per soil type (Reader Geotechniek, 2014)



Zone	*Soil Behavior Type*
1	*Sensitive, fine grained*
2	*Organic soils - clay*
3	*Clay – silty clay to clay*
4	*Silt mixtures – clayey silt to silty clay*
5	*Sand mixtures – silty sand to sandy silt*
6	*Sands – clean sand to silty sand*
7	*Gravelly sand to dense sand*
8	*Very stiff sand to clayey sand**
9	*Very stiff fine grained**

** Heavily overconsolidated or cemented*

p_a = atmospheric pressure = 0,1 MPa

Figure D.7: Soil classification chart based on CPT cone resistance and friction ratio (Robertson 1986, 2010)

Soil type			Representative value [a]) of the soil property											
Main type	mix	consistency [b])	γ [c])	γ_{sat}	q_c [d, g])	C'_p	C'_s	$C_c/(1+e_0)$ [g])	C_α [f])	$C_{sw}/(1+e_0)$ [g])	E_{100} [g, h])	φ' [g])	c'	c_u ($= f_{undr}$)
			kN/m³	kN/m³	Mpa	-	-		-	-	Mpa	°	kPa	kPa
gravel	slightly silty	loose	17	19	15	500	∞	0,0046	0	0,0015	45	32,5	0	-
		moderate	18	20	25	1000	∞	0,0023	0	0,0008	75	35	0	-
		solid	19 20	21 22	30	1200 1400	∞	0,0019 0,0016	0	0,0006 0,0005	90	37,5 40	0	-
	greatly silty	loose	18	20	10	400	∞	0,0058	0	0,0019	30	30	0	-
		moderate	19	21	15	600	∞	0,0038	0	0,0013	45	32,5	0	-
		solid	20 21	22 22.5	25	1000 1500	∞	0,0023 0,0015	0	0,0008 0,0005	75 110	35 40	0	-
sand	clean	loose	17	19	5	200	∞	0,0115	0	0,0038	15	30	0	-
		moderate	18	20	15	600	∞	0,0038	0	0,0013	45	32,5	0	-
		solid	19 20	21 22	25	1000 1500	∞	0,0023 0,0015	0	0,0008 0,0005	75 110	35 40	0	-
	slightly silty clayey		18 19	20 21	12	450 650	∞	0,0051 0,0035	0	0,0017 0,0012	35 50	27 32,5	0	-
	greatly silty clayey		18 19	20 21	8	200 400	∞	0,0115 0,0058	0	0,0038 0,0019	15 30	25 30	0	-
loam [e])	slightly sandy	weak	19	19	1	25	650	0,0920	0,0037	0,0307	2	27,5 30	0	50
		moderate	20	20	2	45	1300	0,0511	0,0020	0,0170	3	27,5 32,5	1	100
		solid	21 22	21 22	3	70 100	1900 2500	0,0329 0,0230	0,0013 0,0009	0,0110 0,0077	5 7	27,5 35	2,5 3,8	200 300
	greatly sandy	-	19 20	19 20	2	45 70	1300 2000	0,0511 0,0329	0,0020 0,0013	0,0170 0,0110	3 5	27,5 35	0 1	50 100
clay	clean	weak	14	14	0.5	7	80	0,3286	0,0131	0,1095	1	17,5	0	25
		moderate	17	17	1	15	160	0,1533	0,0061	0,0511	2	17,5	5	50
		solid	19 20	19 20	2	25 30	320 500	0,0920 0,0767	0,0037 0,0031	0,0307 0,0256	4 10	17,5 25	13 15	100 200
	slightly sandy	weak	15	15	0.7	10	110	0,2300	0,0092	0,0767	1,5	22,5	0	40
		moderate	18	18	1.5	20	240	0,1150	0,0046	0,0383	3	22,5	5	80
		solid	20 21	20 21	2.5	30 50	400 600	0,0767 0,0460	0,0031 0,0018	0,0256 0,0153	5 10	22,5 27,5	13 15	120 170
	greatly sandy		18 20	18 20	1	25 50	320 1680	0,0920 0,0164	0,0037 0,0007	0,0307 0,0055	2 5	27,5 32,5	0 1	0 10
	organic	weak	13	13	0.2	7.5	30	0,3067	0,0153	0,1022	0,5	15	0 1	10
		moderate	15 16	15 16	0.5	10 15	40 60	0,2300 0,1533	0,0115 0,0077	0,0767 0,0511	1 2	15	0 1	25 30
peat	not preloaded	weak	10 12	10 12	0.1	5 7.5	20 30	0,4600 0,3067	0,0230 0,0153	0,1533 0,1022	0,2 0,5	15	1 2,5	10 20
	moderately preloaded	moderate	12 13	12 13	0.2	7.5 10	30 40	0,3067 0,2300	0,0153 0,0115	0,1022 0,0767	0,5 1,0	15	2,5 5	20 30
variation coefficient			0,05		-	0,25						0,10	0,20	

[a]) The table gives the low and the high characteristic value of the average of the soil type concerned. If an increase of the characteristic value of a soil property would lead to a situation that is more unfavourable than the given low value for that property, the value on the right should be used. If there is no value mentioned on the right side of a cell, then the value just below it should be used. This is, for example, the case for negative friction on a pile where a higher value for φ', c' and c_u also results in a high value of the negative friction. The table gives the high characteristic average values for $C_c/(1+e_0)$, C_α and $C_{sw}/(1+e_0)$.
[b]) Loose: $0 < R_n < 0{,}33$; moderate: $0{,}33 < R_n < 0{,}67$; solid: $0{,}67 < R_n < 1{,}00$.
[c]) The γ-values are applicable to a natural moisture content
[d]) The values for q_c (cone resistance) given in this table should be considered as entry values for use of the table and should not be used in calculations.
[e]) The values concern saturated loam
[f]) The C_α-values are valid for a trajectory of stress increase of at least 100%.
[g]) For gravel, sand and to a lesser extent also for loam and sandy clay, q_c, E_{100}, φ' and the compressibility coefficients C'_p, $C_c/(1+e_0)$ and $C_{sw}/(1+e_0)$ are normalised for an effective soil stress σ'_v of 100 kPa. In that sense the equation $q_{c,table} = q_{c,measured} \cdot C_{qc}$ should be used, where $C_{qc} = (100/\sigma'_v)^{0,67}$. For an angle of internal friction φ' and cohesion c' it applies that these are dependent on the consistency of the soil. This implies that this conversion is also needed for φ' and c'. If $q_{c,table}$ would become larger than the value given in the table, the value given in the lowest row of the concerning soil type should be used.
[h]) The Youngs' modulus in case of recurrent stress can be considered to be three times the given value.

Example: In clean sand at a depth of 5 m below water it is measured that: $q_{c,field} = 9$ MPa and $\sigma'_v = 50$ kPa. C_{qc} then is $2^{0,67} \approx 1{,}6$ and $q_{c,table} = 9 \cdot 1{,}6 = 14.4$ MPa. This means that $E = 45$ MPa, $\varphi' = 32{,}5°$, $C'_p = 600$, $C_c/(1+e_0) = 0{,}0038$ and $C_{sw}/(1+e_0) = 0{,}0013$.

Figure D.8: Indicative soil properties according to Eurocode 7 NEN-EN1997 (M. Voorendt, 2023

D.2.1. Data location 1

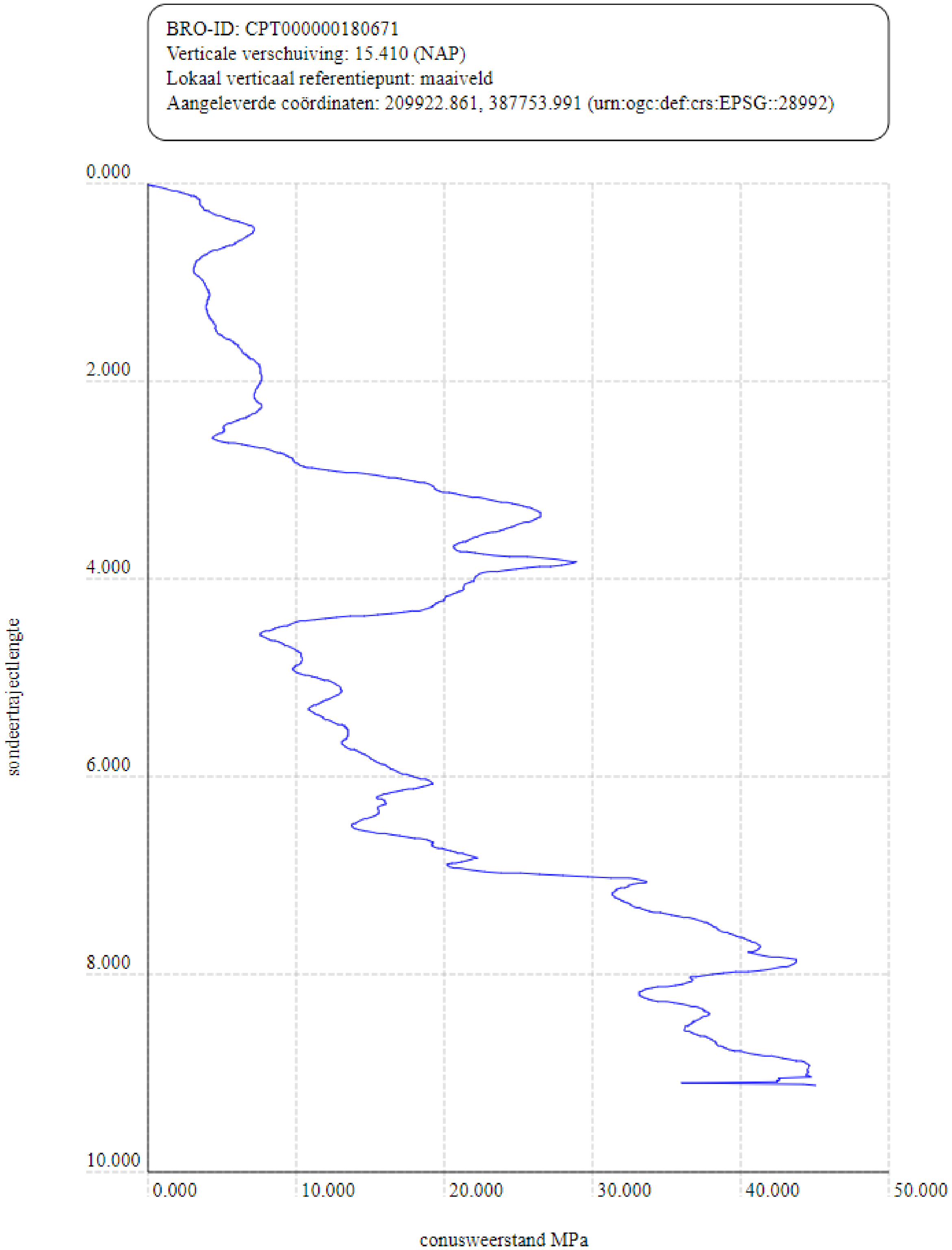


Figure D.9: CPT graph of data location 1 (DINOloket, 2023)

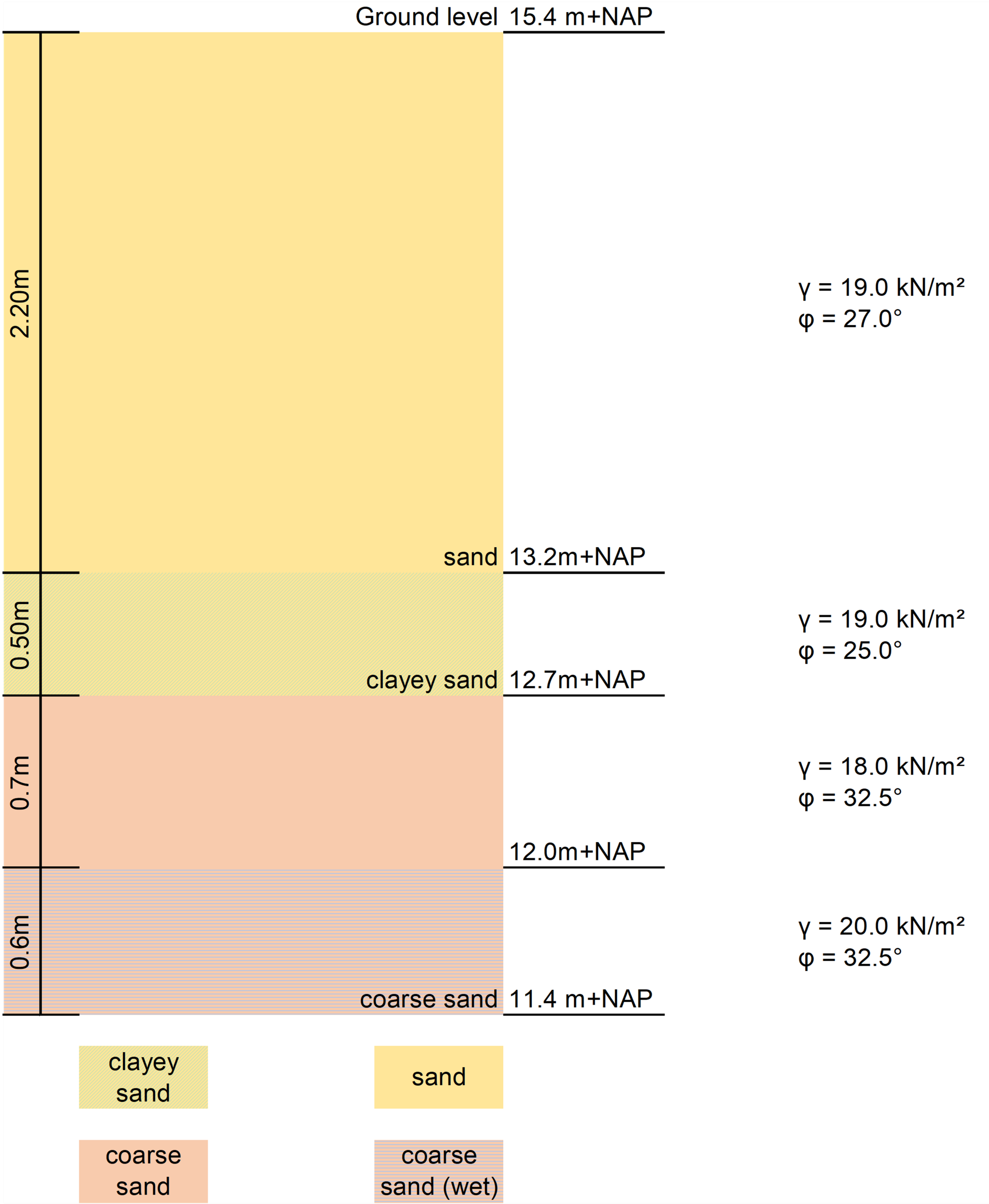


Figure D.10: Reconstructed soil profile for data location 1

D.2.2. Data location 2

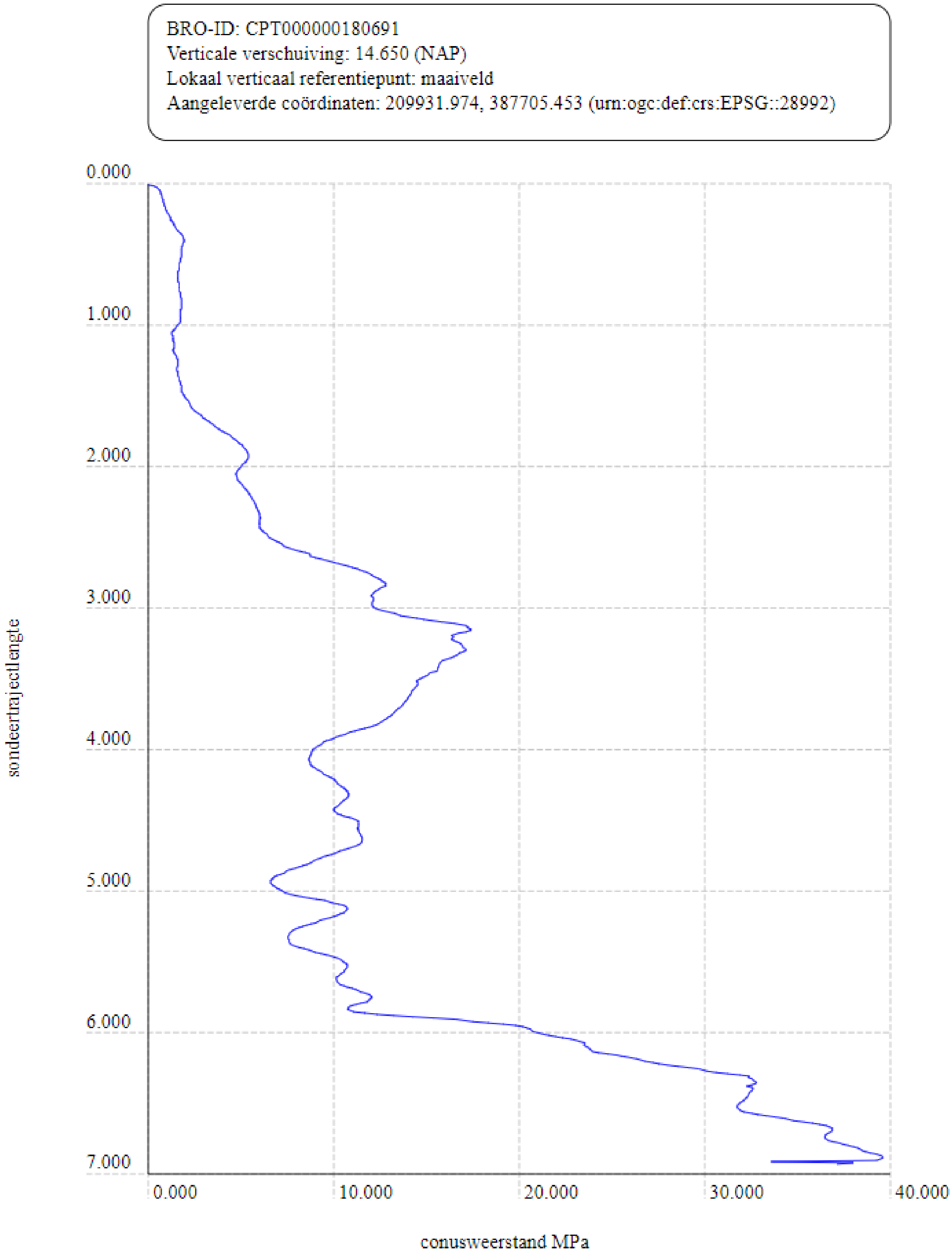


Figure D.11: CPT graph of data location 2 (DINOloket, 2023)

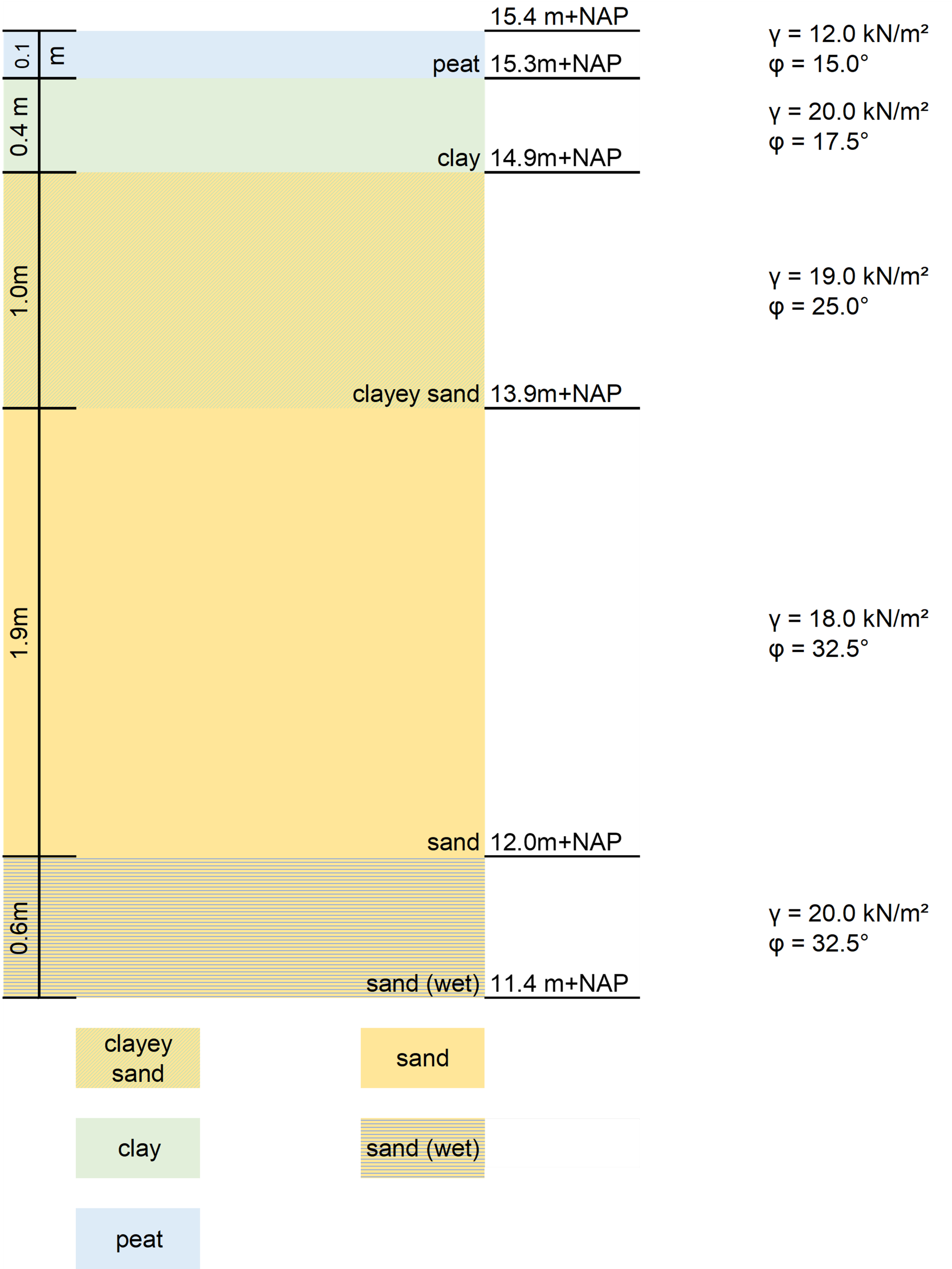


Figure D.12: Reconstructed soil profile for data location 2

D.2.3. Data location 3

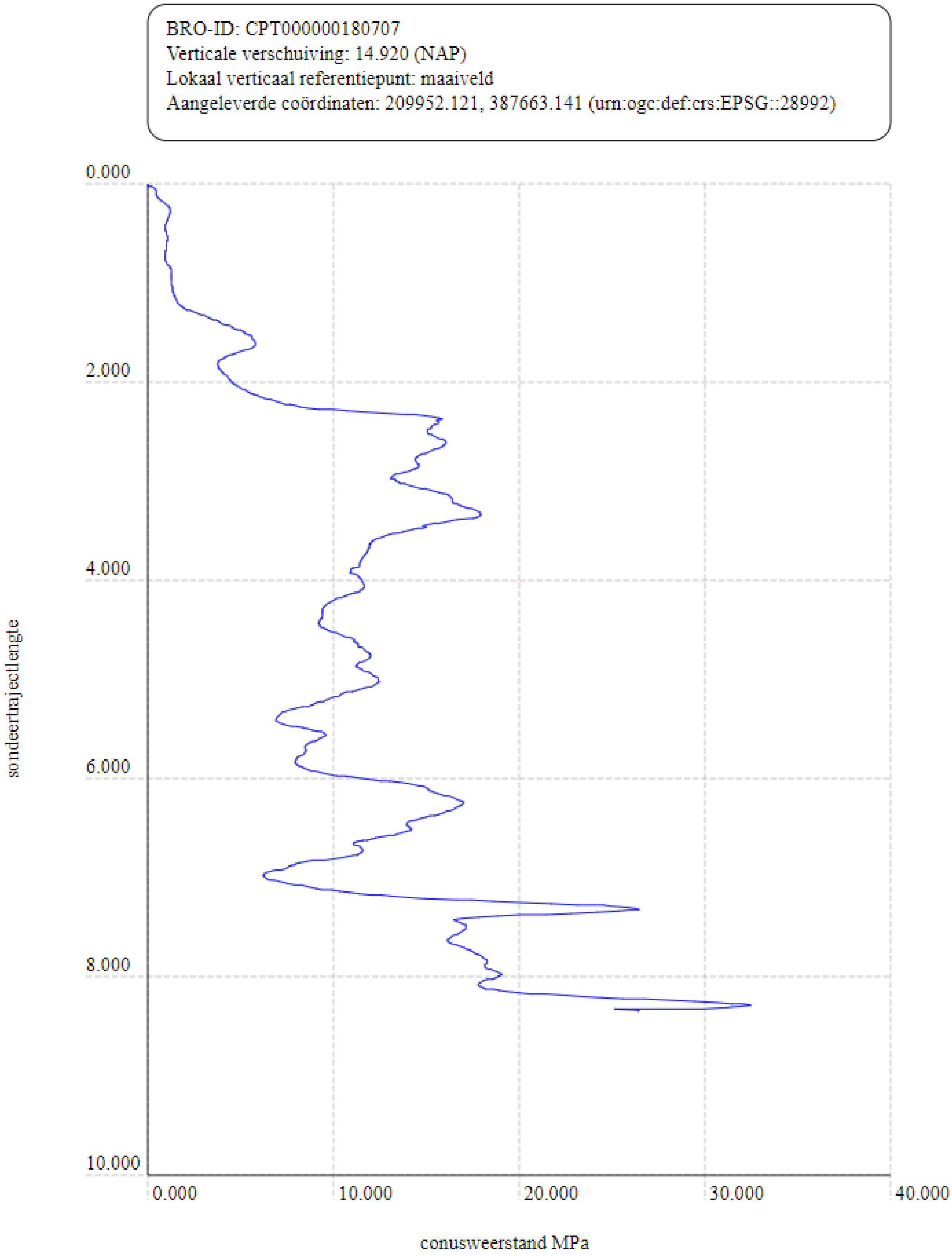


Figure D.13: CPT graph of data location 3 (DINOloket, 2023)

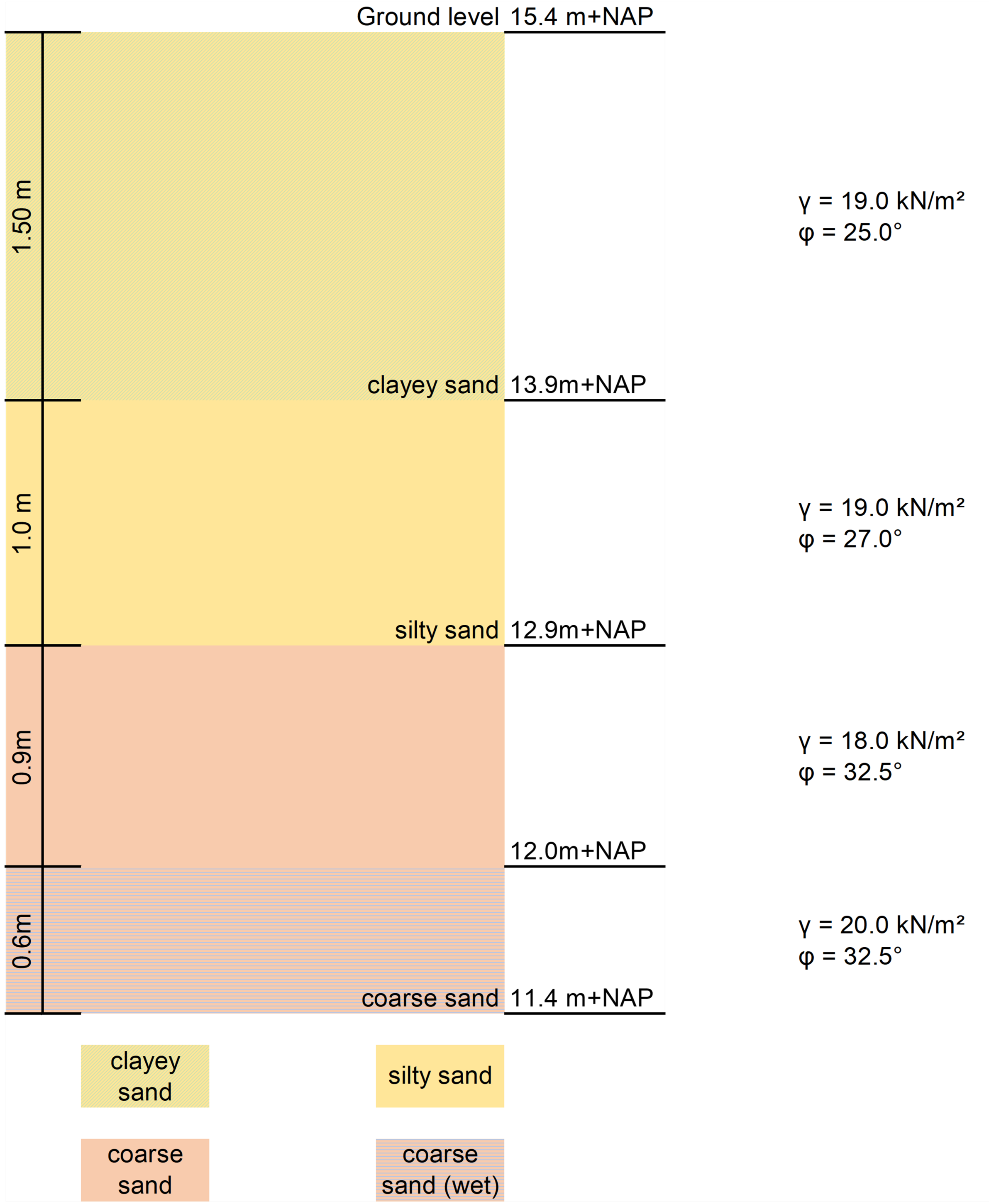


Figure D.14: Reconstructed soil profile for data location 3

D.2.4. Data location 4

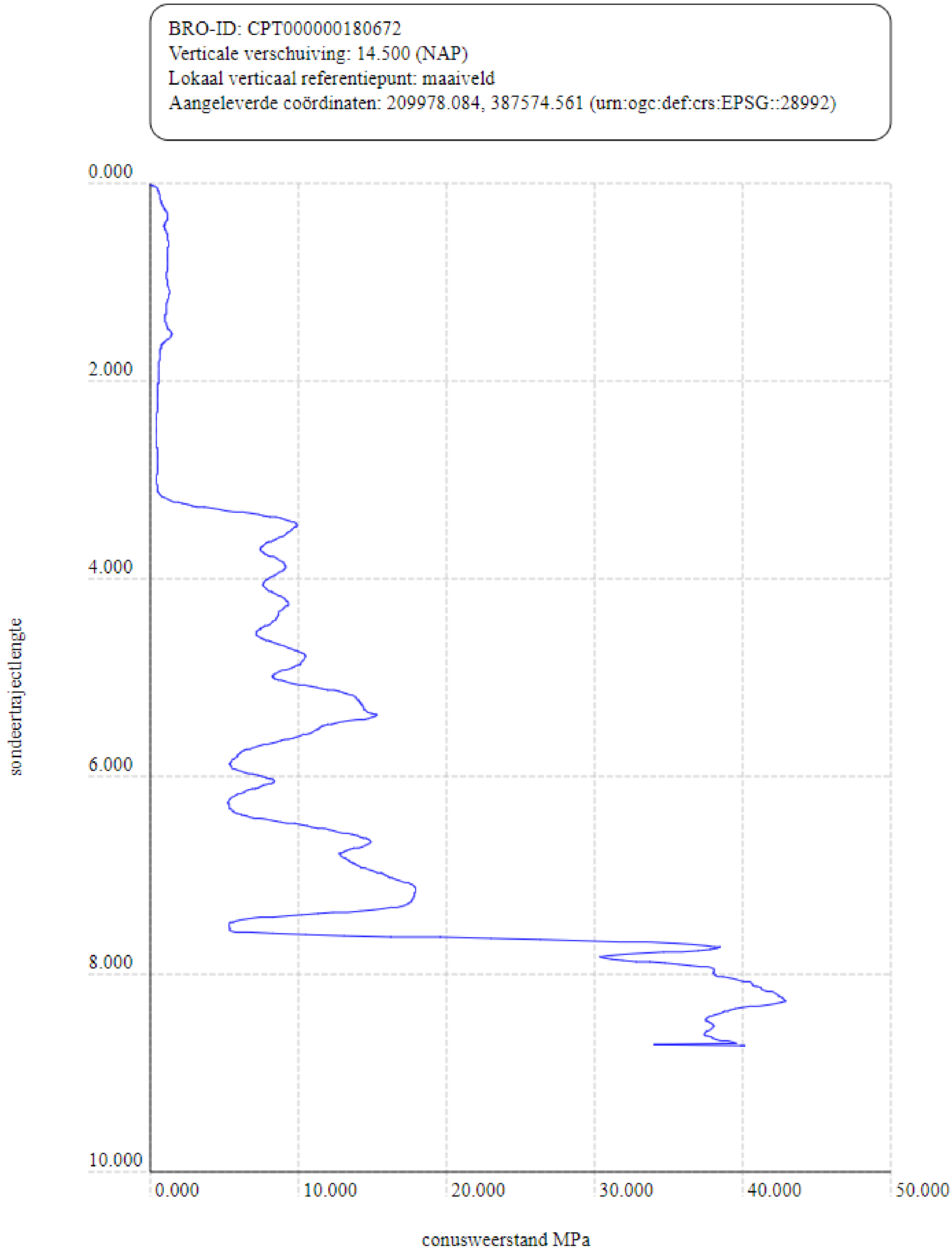


Figure D.15: CPT graph of data location 4 (DINOloket, 2023)

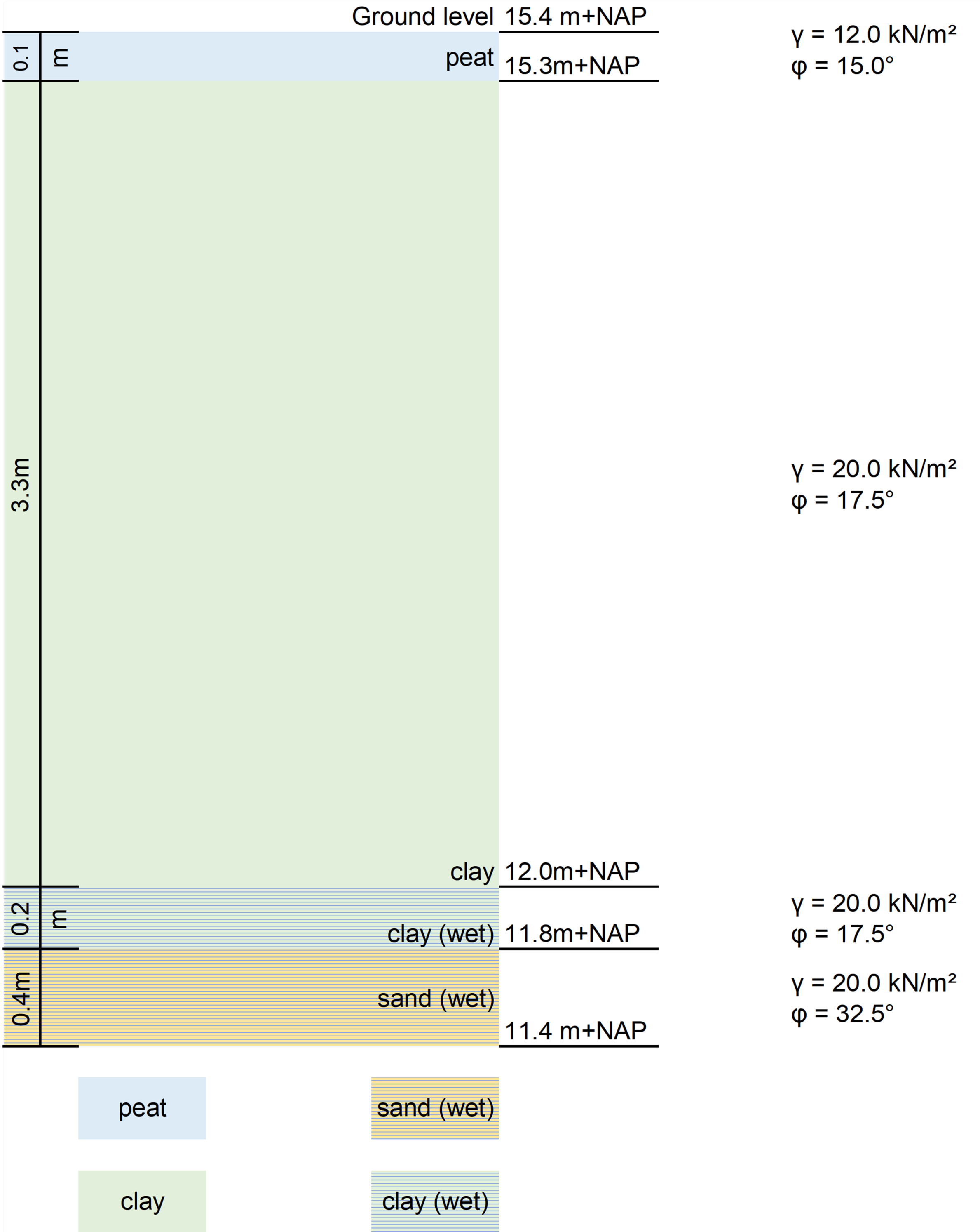


Figure D.16: Reconstructed soil profile for data location 4

D.3. Scoring methods for MCA

This section presents the grading scale for the performance of locations and barrier types on associated criteria.

D.3.1. Location criteria

The locations will be graded based on the four following criteria.

- Disruption ecology
- Spatial quality
- Efficiency
- Length of dike stretch

Disruption ecology
The selection of the location could affect present nature and ecology in the area. The more area of the structure coincides with nature and ecology, the lower the score is given with regard to this criterion.

Spatial quality
The project location area is densely built which means that the structure, depending on the selected dike segment for the exact location, may cross home gardens, touristic areas or general properties of business owners and residents living along the Meuse. This means that during construction at such a location the residents will endure construction hindrance, such as nuisance, unavailability of the construction area etc. The space for the construction area will be more limited if the location is near residential buildings. Because the area is also owned by the residential owners, they are important stakeholders with whom good terms must be agreed upon making the construction process longer and more difficult. Furthermore, if were to be chosen for a location crossing properties, it would mean that part of that land would be unprotected in case of an extreme flood situation for which the structure is built. More specifically, this means that future development plans of the owners for that area will be affected. Besides construction, the structure needs to have yearly maintenance in order to ensure functionality. This means that the residents at least once a year are affected by the structure if the selected dike segment is at the location of the gardens or business properties.

According to the aforementioned explanation of spatial quality, the grading of the locations with respect to spatial quality is performed in a way that relatively less construction hindrance, effects of yearly maintenance and possible loss of land property leads to higher score and vice versa.

Efficiency
If less social challenges to consider are present to achieve the desired result, the efficiency increases. If the amount of stakeholders or social factors, to take into account, increase with a location, the efficiency decreases. For example, locating a structure at gardens, other land properties or touristic areas will raise the need for making arrangements with the stakeholders. If an alternative location could be chosen with less societal issues, the decision-making process goes more quickly. This could be at the cost of other factors, such as the length of the dike segment.

Length of dike section
The larger the stretch of the structure is required at a particular location of the dike section, the less preferable it is to choose for that location, because a longer stretch means higher costs and duration of construction.

In Table D.1 the grading scale for the performance of locations for the self-closing flood barrier on the criteria is shown.

Table D.1: Grading scale for location criteria

Score	Performance for criteria
0	Negative impact for ecology and spatial quality low for efficiency and the longest regarding the length of the dike section
1	Normal impact for ecology and spatial quality, neutral efficiency and equal lengths of the dike sections
2	No negative impact for ecology and spatial quality, high for efficiency and the shortest regarding the length of the dike section

D.3.2. Criteria for barrier type

Adaptibility

The level of adaptivity of the self-closing barrier to increase the water retaining height determines the grading of the concept with the respect to this criterion. The easier an adaptive design could be made, the higher the score is given. For example, for an arched shape wall, it is more difficult to increase the retaining height than extending a flat wall. Also, if more components besides the retaining wall are affected by adapting the water retaining height, the structure is graded with a lower score regarding this criterion.

The scoring method is through grading from 1 to 5, where:

- 1 : Hard to adapt
- 2 : Moderate to hard adaptibility
- 3 : Moderate to adapt
- 4 : Easy to moderate adaptibility
- 5 : Easy to adapt

Integrability

The barrier will be integrated in the area and possibly in the surrounding structures. Each barrier type has a certain extent of integrability in the surrounding area and structures. The more it integrates nicely without impact on the area and without adjustments to the gate, the more preferable the barrier type is. If a certain barrier type requires additional measures to integrate it in the area in comparison with an other barrier type, the grading is done with respectively a lower score. Also, the amount of space occupation across the width determines the score regarding the integrability

The scoring method is through grading from 1 to 5, where:

- 1 : Difficult to connect with area ; space occupation of > 4 m
- 2 : Moderate to difficult connection with area ; space occupation of 3 - 4 m
- 3 : Moderate to connect with area ; space occupation of 2 - 3 m
- 4 : Easy to moderate connection with area ; space occupation of 1 - 2 m
- 5 : Easy to connect with area ; space occupation of > 1 m

Maintainability

The level of complexity of the structure determines the need for maintenance. The larger the amount of elements and connections, the more maintenance is required. This is also true if the structure type is generally larger in size. Furthermore, the access to perform inspection and maintenance plays a major role in this review.

The scoring method is through grading from 1 to 5, where:

- 1 : Difficult to access ; relatively large and many elements
- 2 : Moderate to difficult to access ; normal to large amount
- 3 : Moderately accessible ; relatively normal amount of elements
- 4 : Easy to moderate to access ; few to normal amount of elements
- 5 : Easy to access ; relatively less and small elements

Efficiency
The efficiency relates to the amount of material usage that is required and the functional simplicity of the structure and foundation. If a massive and heavy foundation is required the efficiency goes down and vice versa. The functional simplicity focuses also on the installation of the gate and the complexity of the structure.

The scoring method is through grading from 1 to 5, where:

- 1 : Relatively high material usage ; extreme functional complexity
- 2 : Relatively normal to high material usage ; slight functional complexity
- 3 : Relatively normal material usage ; proximate functional simplicity
- 4 : Relatively low to normal material usage ; slight functional simplicity
- 5 : Relatively low material usage ; extreme functional simplicity

E

Selection of project location

This appendix shows the full elaboration of the selection procedure for the location of the self-closing flood barrier. This involves firstly the verification of the project locations per each dike section against the site requirements. If a location meets the requirement, it is indicated with a green tick in each table and otherwise with a red cross. Secondly, if necessary, a follow-up evaluation is performed resulting in a final selection based on criteria. In Figure E.1 below, a map of the dike segment is shown in which the possible tracks for the placement of the barrier in each dike section are shown indicated with a red and yellow dotted line. The blue dashed lines mark the borders of each dike section and the solid orange line indicates the current retaining structure as part of the current dike segment. The area under consideration begins in the north with the junction between Maasstraat and Broekhuizerweg and ends in the south with the national monument 'The Schans'. At these landmarks the dike segment continues with grass dikes. These areas of Arcen are not considered in this thesis and indicate merely the begin and end of the self-closing flood barrier.

Figure E.1: Project area with dike sections indicated

E.1. Dike section 1

For the first dike section, the top area is considered, where the grass dike from the northern part of Arcen ends and 'Burgemeester Linders-promenade' begins (indicated with the green triangle in Figure E.1. For this top area could be chosen from two possible dike sections. These alternatives are shown in Figure E.1. The yellow dotted line represents the location which follows a walking trail next to the bank of the Meuse. The average elevation along this trail is NAP + 14 m. The dike section has a total length of 350 m, including the top cornered end of 50m connecting with the grass dike. The walking trail has a width of approximately 2 m. In Figure E.2 the walking trail is shown. The red dotted line represents the other possibility for the dike section, which connects directly to the grass dike as is shown with the red circle in Figure E.2. It has a total length of 290 m and an average elevation of NAP + 15 m. The section crosses the private gardens of residents.

Figure E.2: Imagery of possible location for dike section 1 indicated with red (Google Maps, 2023)

E.1.1. Verification against requirements

In Figure E.3 the two possibilities for the location are verified to the requirements.

As can be seen both locations comply and therefore a follow-up evaluation step is done in which the possible locations are scored on criteria, which results in one with the most potential for this project.

Site requirements	Location	
	1	2
Location assures zero buildings involving homes, commerce and hospitality industry or other commercial buildings being excluded from the protected area by Structure.	✓	✓
Location should have a minimum elevation of NAP + 13 m if the length of the dike section in question is less than 50 m, in order to assure a feasible and realistic height of Structure	✓	✓
Location has no obstructions in the soil or does not coincide with present foundations of buildings already built	✓	✓
Location has sufficient space to integrate Structure	✓	✓
Location is available for construction of Structure and is approved by local authorities	✓	✓
Costs for affecting spatial quality factors including important evacuation routes, gardens, protected nature reserves, historical-cultural buildings or monuments, archaeological sites and the ecology in the area, should be compensated for.	✓	✓
Location has necessary adjacent extra works, such as connecting roads and sufficient space to facilitate construction of Structure	✓	✓

Figure E.3: Verification table of locations at dike section 1

E.1.2. Evaluation

In Table E.1 the evaluation is shown. For alternative 1 the ecology is disrupted whereas alternative 2 has a greater impact on the spatial quality. Also, the length of the dike sections differ with 50m. Location 1 is beneficial with respect to efficiency, because with the presence of an alternative location for the dike section, it is not necessary to select location 2 which has more impact on the area. In this evaluation the spatial quality and efficiency are the most important with respect to the other criteria and have therefore a larger weight. The locations are evaluated on the possibility to have the most innovative and creative design for the design.

Table E.1: Evaluation of two locations for dike section 1 based on location based criteria significant to the associated stakeholders (0 = Lowest score ; 1 = Medium score ; 2 = Highest score)

Stakeholder	**Criteria**	**Weight**	**Location**	
			1	**2**
Environmental Impact Assessment committee	Disruption ecology	*1*	0	2
Residents, Contractor, Water Board	Spatial quality	*2*	2	0
Residents, Contractor, Water Board	Efficiency	*2*	1	0
Rijkswaterstaat, Water Board	Length of dike section	*1*	0	1
	Total score		**6**	**3**

E.2. Dike section 2

The second dike section lies in the area displayed in Figure E.1 which has two possibilities to continue, as is shown in Figure E.1. The red dotted line represents a dike section where the structure is integrated in a walking trail where the elevation is approximately NAP + 12 to 13 m. The yellow dotted line represents a dike section that crosses private gardens from residents living along the Meuse, where the elevation is NAP + 15 m. In Figure E.4 imagery of this area is shown.

Figure E.4: Imagery of possible location for dike section 2 (Google Maps, 2023)

E.2.1. Verification against requirements

In Figure E.5 the two possibilities for the location are verified to the requirements.

Site requirements	Location 1	Location 2
Location assures zero buildings involving homes, commerce and hospitality industry or other commercial buildings being excluded from the protected area by Structure.	✓	✓
Location should have a minimum elevation of NAP + 13 m if the length of the dike section in question is less than 50 m, in order to assure a feasible and realistic height of Structure	✗	✓
Location has no obstructions in the soil or does not coincide with present foundations of buildings already built	✓	✓
Location has sufficient space to integrate Structure	✓	✓
Location is available for construction of Structure and is approved by local authorities	✓	✓
Costs for affecting spatial quality factors including important evacuation routes, gardens, protected nature reserves, historical-cultural buildings or monuments, archaeological sites and the ecology in the area, should be compensated for.	✓	✓
Location has necessary adjacent extra works, such as connecting roads and sufficient space to facilitate construction of Structure	✓	✓

Figure E.5: Verification table of locations at dike section 2

For this location, the option indicated in yellow in Figure E.1 is selected, because the elevation at the red alternative is too low, where a lot of levelling with sand or a structure with a too large retaining height is required, which can be very uneconomical. In this thesis the design will be based on compensating costs for the impact on spatial quality rather than extra costs for a larger structural size.

E.2.2. Evaluation

One possibility complies to the requirements, so a following evaluation procedure is not required to make a selection.

E.3. Dike section 3

Dike section 3 involves the part of the flood barrier along the restaurant Alt Arce with its popular touristic 'Maasterras', see Figure E.6.

Figure E.6: Imagery of possible location for dike section 3 (Google Maps, 2023)

This location has an elevation of NAP + 12m and very little space to integrate a structure. The following three possibilities will be considered for this:

- 1. Along the elevated terrace, local elevation of the walking trail with a stairs as shown in the reference image, Figure E.7, in which the self-closing flood barrier is integrated. The elevated surface along the terrace will be continued.

Figure E.7: Reference imagery for elevation of the walking trail with a stairs (Google Maps, 2023)

- 2. Integrate the self-closing flood barrier into the wall of the elevated terrace.
- 3. Intersect the restaurant and terrace with the self-closing flood barrier.

Alternative 1 is an option in which the owner of the restaurant, is influenced the least, because the dike section is located along the terrace. (An important remark with this alternative is that, with placing a stairs, people not able to climb stairs are excluded from walking this trail further. This is a downside with respect to the spatial quality.)

Alternative 2 would involve temporary demolition of the terrace, reinforcing the wall elevating the terrace and integrating the self-closing flood barrier in the wall and reconstructing the terrace. This involves making proper arrangements with the owner and compensation for revenue loss.

Alternative 3 would also involve temporary demolition of terrace, constructing the self-closing flood barrier separating the terrace from the restaurant. This option also requires proper arrangements with the owner. Additionally, the owner plays a major role with the functionality of the barrier in times when it needs to be activated.

E.3.1. Verification against requirements

In Figure E.8 the two possibilities for the location are verified to the requirements.

Site requirements	Location		
	1	2	3
Location assures zero buildings involving homes, commerce and hospitality industry or other commercial buildings being excluded from the protected area by Structure.	✓	✓	✗
Location should have a minimum elevation of NAP + 13 m if the length of the dike section in question is less than 50 m, in order to assure a feasible and realistic height of Structure	✓	✓	✓
Location has no obstructions in the soil or does not coincide with present foundations of buildings already built	✓	✓	✗
Location has sufficient space to integrate Structure	✓	✓	✓
Location is available for construction of Structure and is approved by local authorities	✓	✓	✓
Costs for affecting spatial quality factors including important evacuation routes, gardens, protected nature reserves, historical-cultural buildings or monuments, archaeological sites and the ecology in the area, should be compensated for.	✓	✓	✓
Location has necessary adjacent extra works, such as connecting roads and sufficient space to facilitate construction of Structure	✓	✓	✓

Figure E.8: Verification table of locations at dike section 3

E.3.2. Evaluation

Alternative 3 does not comply to the site requirements regarding exclusion of buildings from the flood protected area and coinciding with present foundations in the area. Alternative 1 seems to be the most effective solution to continue the elevated walking trail along the 'Maasterras' in which the self-closing flood barrier can be integrated, in comparison with alternative 2. In this way, little consideration needs to be given to the owner of the restaurant. Additional demolition and reconstruction are thus saved and the walking trail is not significantly affected other than the elevation. In conclusion, the first alternative will be the location of the self-closing flood barrier for this dike section

E.4. Dike section 4

The following part of the dike section is in the area between the 'Maasterras' and the national monument 'Schanstoren', see Figure E.9. In this area two possibilities are available as are illustrated in E.1. The red dotted line represents a dike section where the structure is integrated in a walking trail directly along the river bank of the Meuse where the elevation is approximately NAP + 12 to 13 m. The yellow option involves integrating the self-closing flood barrier in the embankment between the walking trail and the present walls that separate the boardwalk from the private gardens, which has a slightly higher elevation.

Figure E.9: Imagery of area for dike section 4 (GeoWeb Rijkswaterstaat, 2023)

E.4.1. Verification against requirements

In Figure E.10 the two possibilities for the location are verified to the requirements.

Site requirements	**Location**	
	1	**2**
Location assures zero buildings involving homes, commerce and hospitality industry or other commercial buildings being excluded from the protected area by Structure.	✓	✓
Location should have a minimum elevation of NAP + 13 m if the length of the dike section in question is less than 50 m, in order to assure a feasible and realistic height of Structure	✗	✓
Location has no obstructions in the soil or does not coincide with present foundations of buildings already built	✓	✓
Location has sufficient space to integrate Structure	✓	✓
Location is available for construction of Structure and is approved by local authorities	✓	✓
Costs for affecting spatial quality factors including important evacuation routes, gardens, protected nature reserves, historical-cultural buildings or monuments, archaeological sites and the ecology in the area, should be compensated for.	✓	✓
Location has necessary adjacent extra works, such as connecting roads and sufficient space to facilitate construction of Structure	✓	✓

Figure E.10: Verification table of locations at dike section 4

E.4.2. Evaluation

The track indicated in yellow is selected for the same reason as was done for dike section 2, because the elevation is too low at the other possibility (in red), which is not in line with the requirements. So, one possibility remains, so a following evaluation procedure is not required to make a selection.

E.5. Dike section 5

The last dike section concerns the area with the 'Schanstoren' which has high cultural-historical value, because it is registered as a national monument. For this area two possible locations are available to construct a self-closing flood barrier, indicated with red and yellow in Figure E.1. The red alternative puts the historical tower inside the flood protected area, whereas the yellow alternative leaves the historical tower outside the flood protected area. Important mentioning hereby is that at the location of the red option, the ground level elevation is around 12 m + NAP, whereas the yellow option has a ground level elevation of 15 - 16 m + NAP.

Figure E.11: Imagery of area for dike section 5 (Google Maps, 2023)

E.5.1. Verification against requirements

In Figure E.12 the two possibilities for the location are verified to the requirements.

As is the case for the other dike sections, the red option has a too low ground level elevation, which would result in a very high flood barrier and this does not comply to the site requirements. This is not feasible and thus, the selected location for this dike section is the green alternative. This means that the historical tower will be put outside the flood protected area.

Site requirements	Location	
	1	2
Location assures zero buildings involving homes, commerce and hospitality industry or other commercial buildings being excluded from the protected area by Structure.	✓	✓
Location should have a minimum elevation of NAP + 13 m if the length of the dike section in question is less than 50 m, in order to assure a feasible and realistic height of Structure	✗	✓
Location has no obstructions in the soil or does not coincide with present foundations of buildings already built	✓	✓
Location has sufficient space to integrate Structure	✓	✓
Location is available for construction of Structure and is approved by local authorities	✓	✓
Costs for affecting spatial quality factors including important evacuation routes, gardens, protected nature reserves, historical-cultural buildings or monuments, archaeological sites and the ecology in the area, should be compensated for.	✓	✓
Location has necessary adjacent extra works, such as connecting roads and sufficient space to facilitate construction of Structure	✓	✓

Figure E.12: Verification table of locations at dike section 5

E.5.2. Evaluation

One possibility complies to the requirements, so a following evaluation procedure is not required to make a selection.

E.6. Summary

In Figure E.1 the yellow dotted line represents the entire track of the location of the self-closing flood barrier.

F

Background information on concepts for hydraulic gates

F.1. Structural types of hydraulic gates

F.1.1. Flat gate

General information

The flat gate is the simplest way to fulfil a water retaining function. The reason for this is because of the linear movement which corresponds with a relatively short and simple way of gate closure (Daniel & Paulus, 2018). The hydraulic load transfer is by bending (Daniel & Paulus, 2018), which is different for, for example a radial gate that uses only strut forces. The downward movement of this gate, in order to 'open' the barrier is exerted by gravity (Daniel & Paulus, 2018). The closure of the gate, however, does need an external drive. The gate leaf most of the time consists of a plate in the vertical plane with horizontal and vertical stiffeners if necessary, which in fully lowered position rests in a concrete chamber. The gate could be executed with synthetic materials as a composite structure or mainly in steel for example. A flat gate system in this design project, does not have piers above ground level to transfer the forces to, because of the requirement of preserving the townscape. This means that over the entire span, large moments are caused that are directly transmitted to the concrete chambers embedded in the soil. This will lead to the need for special attention for the anchoring which is less of a problem with other gate types.

Evaluation criteria based

1. Adaptibility

The flat gate is the most simple gate type to adapt for a higher water retaining level. The skin plate can be easily extended and the foundation has a minimal impact from this if an additional free height is saved inside the chamber in which an extended gate also fits. For a flap gate, this is also rather easy but an additional concrete chamber should be added to house the extra part of the gate.

2. Integrability

A flat gate can be integrated in the area by embedding the concrete housing in the subsoil. It has the advantage with respect to the integrability because of the small width of the structure resulting from the fact that it linearly moves compared to a gate with a rotational movement. However, as was mentioned earlier, the foundation needs additional attention because it should directly resist the moments caused by the hydrostatic loads. Additionally, the depth of the foundation is larger, which results from the fact that it should house the gate leaf that has a length equal to the entire retaining height. Moreover, the system has a relatively small width, which is not favorable when resisting large moments. Hence, heavy anchoring should be considered, which means that extra foundation works should be done in the area, affecting the spatial quality during construction.

3. Efficiency

Because the skin plate should resist global bending, the material usage is higher than for example with a radial gate. However, this problem can be countered by using lightweight materials for the gate leaf. The gate has a high functional simplicity because of the linear motion of the gate. Rotational gates require more movable rotating elements, which makes the structure more complex.

4. Maintenance
Inspection and maintenance is relatively easy if the gate is lifted. The embedded concrete structure however is more complicated to inspect and maintain (Daniel & Paulus, 2018). The inside of the chamber where the guides and components responsible for the driving mechanism are, are hard to access, because of the small width involved. Maintenance costs are relatively low.

F.1.2. Flap gate

General information

The flap gate is executed with a skin plate which can be either straight or curved. The skin plate is hinged to the sill ensuring a rotational movement around a fixed horizontal axis. Depending on the shape of the leaf and the retaining height, the spans can go up to 20 to 100 m. This gate type can be executed with two skin plates in order to obtain a wide span. The longest flap gate ever built has a span of 100 m and a height of 3.7 m which is in St. Pantaleon, Austria (Daniel & Paulus, 2018). It is common for this type of gate to have a maximum angle of rotation of 60 to 70 degrees (Daniel & Paulus, 2018). For instance, this means that the height of the leaf should be minimally around 2.5 m if the retaining height is 2 m. In fully lowered position the leaf can be integrated nicely within the surface. Depending on the drive mechanism, this gate type needs a recess in the concrete foundation to house the machinery or elements responsible for the drive system.

Evaluation criteria based information

1. Adaptibility
The flap gates can be adapted to retain a larger water height by extending the skin plate. The original leaf does not have to be fully replaced. For a water retaining barrier acting as flood protection as part of a dike section, the embedment of the leaf is not as deep as for a flat gate for example. For this reason, adapting the gate is easy because the gate is relatively accessible.

2. Integrability
Since the embedment of the leaf is relatively shallow near ground level, it can be integrated nicely with the surface area. However, since it has an angled position when fully rotated, adjacent flap gates from different dike sections that may not be in a single straight line need more attention with regards to a water tight sealed connection. Furthermore, the angled position leads to a larger required height of the leaf. In a fully lowered position the height dimension is projected to the surface as the width, which means that space usage is relatively larger for flap gates than other gate types. In areas with little space this is not preferable.

3. Efficiency
The flap gate has the option to be manufactured with unconventional materials such as synthetics which are cheaper and require less maintenance than for example steel elements. Furthermore, in comparison with other gate types, the foundation width is more or less in line with the width in fully lowered position, resulting in a relatively more stable embedment than with a slender and deep embedment.

4. Maintenance
The gate leaf is accessible for maintenance and inspection when in retaining position. It has the advantage that most of the time the structure has a shallow concrete substructure. However, the hinges are less accessible.

F.1.3. Sector gate

General information

The sector gate is similar to a radial gate, which will be discussed in the next section. However, both gate types have different operations. The sector gate uses hydrostatic upward pressure from the inside surfaces of the skin plates to pivot around a horizontal axis fixed at the sill. The sector gate could also be executed with a completely closed off bottom plate, making it a floating vessel which operates hydraulically as well but from the outward bottom surface (Daniel & Paulus, 2018). This is also known as the drum gate. Another execution of the sector gate is a rotary segment gate where the gate is fixed to a rotary disk which is not hydraulically driven. However, this type uses large piers above ground level in order for the gate to operate (Daniel & Paulus, 2018). This type is mainly used as lock gates which is two sided water retaining. This variant of the sector/radial gate will not be considered further since it does not comply to the functional requirements. Additionally, executing a rotary disk with only a partial circular shape and with the piers fully embedded in the subsoil, would result to a concept complying to the requirements but would just be a gate of the same nature as a drum gate with a different operating mechanism.

For the evaluation phase, only the open sector gate will be considered further, because the drum gate has more vulnerabilities than the open sector gate. For this reason only the open sector gate will be discussed for the following criteria and it will be referred to as ‘sector gate’, i.e. no distinction will be made whether it is open or closed vessel.

Evaluation criteria based information

1. Adaptibility

The sector gate can be extended, but due to the curved shape this requires more attention. Here applies the same as for the radial gate, only there is no question here of adding a strut arm since the sector gate is not equipped with strut arms.

2. Integrability

The sector gate integrates nicely with the surrounding area in fully lowered position because of the straight top plate. However, the gate uses more space than a flat gate as is the case for the radial and flap gate. Since the gate transmits the hydrostatic pressure forces directly onto to the sill, the sill is massive over the entire span (Daniel & Paulus, 2018). This takes relatively long time for construction and results in more construction hindrance. Additionally, considering the alignment of different dike sections, which are not in a straight line, connecting different adjacent sector gate section with each other make the sector gate less suitable regarding the integrability.

3. Efficiency

A sector gate is generally massive and heavily founded over the entire span which is not preferable when it comes to constructing a flood barrier in gardens. However, the material usage for the gate leaf is relatively less, because it is commonly a lightweight structure because it uses buoyancy to raise in retaining position. For the sector gate generally, the installation requires high accuracy, which takes more construction time and hindrance which is not preferable for the people living at the site. Furthermore, rotational gates require more movable parts, which reduces the functional simplicity. (Daniel & Paulus, 2018)

4. Maintenance

The sector gate is maintainable when in fully raised position. However, to raise a sector gate which is characteristically driven by buoyancy, external lifting equipment is necessary like cranes in order to inspect and maintain the gate in the dry. Inspection and maintenance for a sector in fully lowered position is poorly accessible. The amount of maintainable parts is the highest for the sector and flap gate because these gate types require hinge supports over the entire span (Daniel & Paulus, 2018).

F.1.4. Radial gate

General information

The radial gate is a rotational gate about a horizontal axis with a curved skin plate. The gate leaf has the shape of an arc or a segment and is connected to the supports with strut arms that have the same length as the radius of the curvature of the skin plate. The strut arms converge to the point of the rotational axis ensuring that the forces are led to trunnions (Daniel & Paulus, 2018). The trunnions are the hinged supports of the gate and are in the rotational axis of the gate. With this arrangement no moments are caused that should be resisted by the trunnion or the machinery responsible for the driving mechanism. This arrangement makes sure that the resultant force of the hydrostatic pressure acts radially towards the axis of the rotation. However, this is not a strict condition for a radial gate, although eccentricity of the resultant force should be limited since the main characteristic of the radial gate is to allow mainly compressive forces and small moments to occur. Otherwise another gate type would be preferred. A radially acting resultant force means that the trunnions are the heaviest loaded components of the gate and are the focus of the gate system since they collect all hydraulic loads, partially the self-weight and loads resulting from the drive mechanism then transferring these loads to the concrete piers or foundation while also acting as the hinge for the structure (Daniel & Paulus, 2018).

The concrete piers or foundation should thus be well anchored. Radial gates are frequently used as a spillway structure and are primarily constructed with structural steel but other materials are not necessarily excluded. The arrangement of the gate allows it to be mainly subjected to compressive forces. Thus, attention should be given to buckling in the design process. Furthermore, since the trunnions are the most essential components of the system, replacing a subcomponent of the trunnions is difficult and costly (Daniel & Paulus, 2018). Normally radial gates are used as spillway structures (Daniel & Paulus, 2018). However, the radial gate could be applied as a flood barrier as well. In the case of a self-closing flood barrier in Arcen, the gate, struts and trunnion would be lowered in a recess in the foundation.

Radial gates can be executed with buoyancy tanks supporting the strut arms in order to have a buoyant driving mechanism (Daniel & Paulus, 2018). The tanks are in special floatation chambers which can fill up with water. This execution will be more complicated with larger spans, because of the weight of the structure. Also, radial gates can be executed with counterweights. For such an arrangement, the strut arms are extended past the trunnion to connect the counterweight. The larger the radius of the skin plate curvature and the higher the bearing, the higher will be the lifting required. The radius is often 1 – 1.2 times the gate height. The smaller the radius the heavier the gate.

Evaluation criteria based information

1. Adaptibility

The curved skin plate of the radial gate can be extended to attain a higher water retaining level. This requires more work than for example a flat or flap gate because of the curvature of the skin plate. If a straight plate extension is considered, the force distribution will be different and not characteristic anymore for a radial gate. Moments are then able to develop. Furthermore, curved extension of the skin plate is only possible if the radius is larger than the retaining height. Also, because the radial plate is supported by strut arms, adapting the gate leaf may lead to the need for an extra strut arm (Daniel & Paulus, 2018). In conclusion, the adaptability for a radial gate is harder compared to other gates.

2. Integrability

The radial gate has an advantage with respect to integrability because of the large possible spans. However, too large spans are not desirable with respect to the consequential damages in case of system failure. Large possible spans would be advantageous regarding excavation for the intermediate supports. However, for a radial gate the excavation over the entire span is still equal to the gate height, because the gate leaf should be housed in the recess of the concrete foundation. Thus, this means that in comparison with a flap gate and sector or drum gate the difference in excavation is not large. The difference lies in constructing the concrete bearings which, for a radial gate is only at the sides of the spans. But still for this gate type, the area will be affected anyways with respect to construction hindrance. Moreover, the anchoring of the trunnions are extensive due to the heavy loading (Daniel & Paulus, 2018), so construction works at those locations will take longer than the rest of the gate.

Large spans also have an additional attention point with respect to the balanced lifting of the gates, because the larger the span, the harder balancing gets when lifting the gates. So large possible spans are still not considered a large advantage over the other gate types.

A radial gate as a flood barrier, which should be concealed in the subsoil when in fully lowered position, requires some sort of cover which integrates with the landscape. This is already the case for the other gate types, because of the arrangement of the gate leaf. For radial gates this is a problem because without a cover it would be fully visible and since the location of the gate crosses through gardens and should integrate nicely with the area, this is not practical and safe. The gate leaf could be executed with a plate to cover the structure that rotates along with the rest of the gate. However, this results in more construction works and additional material in comparison to the other gate types where the integrability in the area is already ensured because of the shape of the gate leaf.

3. Efficiency
The material usage is relatively low, because of the advantageous force directing. Radial gates in general are, however, mainly used for spillway structures and design problems with a large differential head. For this reason, it is often executed in steel. This does not necessarily mean that it could not be used as a flood barrier on foreland or that it could not be executed with unconventional materials. But it requires some additional thought about the efficacy. It has a lower functional simplicity than a flat or flap gate. For a spillway structure this comparison would be different.

4. Maintenance
Because of the gate arrangement, the gate leaf can be rather lightweight executed. This is beneficial with respect to maintenance. Also, large part of the maintenance involve the trunnions. These are more accessible than the hinge of a sector gate or the slides and guides of the flap gate.

F.2. Drive mechanisms for hydraulic gates

F.2.1. Hydraulic cylinder

Cylinders are used to produce a force in order to move an object in a linear motion. The force can be generated in various ways such as:

- Hydraulic - An incompressible fluid is pressurized which delivers the power to produce the force. This produces generally high forces.
- Electric - An electric motor generates a rotary motion which converts into a linear motion by a flexible connection or a worm gear transmission. This generally delivers a precise motion.
- Pneumatic - A gas is compressed that delivers the power to produce the force. This generally produces a high speed motion.

Thus, various options are available for a direct cylinder driven motion for a self-closing flood barrier, but for an hydraulic structure an hydraulic cylinder is the most suitable because of the high force motion. Only the hydraulic cylinder will be considered further in this design project.

Hydraulic cylinders are widely used for different gate types. An hydraulic cylinder consists of mainly:

- piston rod which is the outward moving rod to transmit the linear motion
- piston
- seals for the piston rod and the piston to prevent leakage
- cylinder tube in which the hydraulic fluid is pressurized
- two flow ports for inflow and outflow of the hydraulic fluid
- steel barrel that functions as the casing for the cylinder

The piston rod is most of the time chrome plated and requires little maintenance. Seals on the other hand have to be replaced from time to time. Hydraulic cylinders come in different types, from which one of them is double action which means that it extends and retract which is convenient for gate closure. A commonly used type is the mill-type cylinder because it delivers the highest pressure. For a cylinder, buckling is the most critical in the design. Cylinders with a larger rod diameter should be chosen if they are loaded in compression. Hydraulic cylinders are activated by hydro power units which need to be stored.

Cylinders are favored above wire ropes, chains and gear racks because of the following reasons:

- Cylinders have reliable seals and piston rod coatings that prevent leakage of drive oil
- Cylinders need less structures and components than mechanical drives
- There is no lubrication required, because of better wear and corrosion protection in comparison to mechanical drives
- Cylinders require low energy consumption relative to the other mechanisms
- Cylinders have better conditions for remote control and maintenance. However, the placement of the cylinder is important factor in this, because if its integrated inside the gate in case of for example a flat gate. Then, maintenance will be more difficult because of the poor access to it.

F.2.2. Wire rope

Wire ropes are steel cables that can hoist gates with tensile forces. They are winched around drums that can rotate. The rotational movement of these drums is driven by hydraulic cylinders or electomechanical drives such as gears with a rod. For this a hydro power unit or electric motor is required.(Daniel & Paulus, 2018)

Wire ropes are often round in diameter, comprising of steel wired strands helically twisted around the the centre line of the rope. Wire ropes need close attention for the diameter and the diameter of the drum. Typically a ratio of 30:1 or higher are used. Otherwise, the wire rope can wear easily, experience fatigue and thus ultimately fail. Wire rope diameters vary commonly from 22 mm to 38 mm, meaning drum diameters of 660 to 1140 mm should be taken into account in the space usage. For wire ropes typically a minimum safety factor of 5 is used in gate drives and lifting.(Daniel & Paulus, 2018)

Advanced wire rope technology resulted in superseding of chain hoists. Chain hoists deal with various maintenance problems. For this reason chains are omitted in this outline of drive mechanisms. Wire ropes are a clear choice when the submergence of the gate is large, where hydraulic cylinders do not have the stroke capacity or when the loads are too large for a hydraulic cylinder. Wire rope systems are more suitable for structures that are infrequently used than hydraulic drives.(Daniel & Paulus, 2018)

To prevent corrosion, since the material of wire ropes is steel, lubrication is required for galvanised wire ropes and stainless steel wire ropes. Galvanised wire ropes have a reduced bending capacity. Stainless steel wire ropes have lower strength, so often larger diameters are required ore more ropes. Regular inspection is required for a stainless steel wire rope but it does have resistance to corrosion in contrast to a galvanised wire rope. So stainless steel wire ropes are often used where inspection and lubrication is difficult or where contact with water can not be ruled out.(Daniel & Paulus, 2018)

In Figure F.1 an example of a wire rope system is shown.

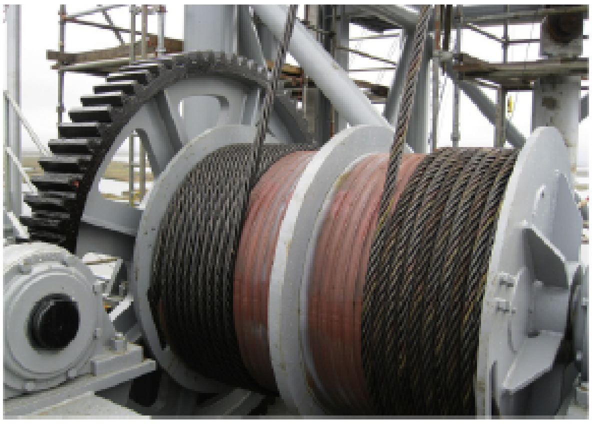


Figure F.1: Example of a wire rope system winched around a drum (Daniel & Paulus, 2019)

F.2.3. Buoyancy driven

Objects can stay afloat in water because of a buoyant force. Buoyancy is based on the Archimedes' principle where an object that is partially or totally immersed in fluid is subjected by a force equal to the weight of the displaced fluid volume. When this buoyant force is equal to the weight of the object, the object stays afloat. Knowing this, it can be observed that increasing the water level in a certain water body, also results in a displacement of the object. Observing this process over a certain time span, one can say that the object moves and is driven by buoyancy. Now more specifically, if the object is a hydraulic gate, the gate can be set in motion by intentionally inducing buoyancy on the gate. Herein, the material of the gate plays a major role. For example, with a steel gate the volume per unit meter span or the cross-sectional area needs to be 7.85 times smaller than the amount of displaced water. The reason for this is that the barrier is made from steel and steel has a specific weight of 7850 kg/m³ whereas water has a specific weight of 1000 kg/m³ which means that steel has a 7.85 larger weight which must be compensated for in the amount of displaced water for a floating object to receive equilibrium, hence floating. Thus, in order for the barrier to float, sufficient cross-sectional area must be provided which displaces the water. The displacement of water is often provided with so-called buoyancy chambers or air tanks. The chambers are required to be water tight welded and to be inspected on a regular basis to ensure the water tightness of the air tanks.

Other characteristics of a buoyancy driven systems are:

- Stability is important with buoyancy, both during movement, which involves the static floating stability and in the retaining position, i.e. the end of the movement, where buoyancy should be well balanced to obtain force equilibrium on the gate. Otherwise overrotation can occur.
- Buoyancy reduces- the energy consumption with respect to other drives.
- Buoyancy driven systems are almost free of mechanical parts, which imparts a certain simplicity to the system and reduces the maintenance.
- Because of the fact that it can be fully autonomous it is not a labor intensive mechanism.
- When the material of the gate has a much larger specific weight than water, such as with steel, it may be possible depending also on the hydraulic boundary conditions, that the air tank or floatation chamber needs to be generously dimensioned.
- Depending on the gate type, only part of the gate weight need to be balanced with buoyancy. This is the case with flap gates, where a part of the self-weight is directed to the hinge. For flat gates the entire gate weight needs to be balanced.

F.2.4. Inflation

A gate drive with inflation means inducing motion to the gate with the help of inflatable bladders. A good example of this is the Obermeyer gate system. The bladders or bellows get inflated by air supply from a pressure vessel and blowers or compressors, by water supply or by a mix of air and water. When the bladder inflates it increases in volume, which means that it induces a motion to the gate when the gate is attached to such a bladder. with the required inside pressure in the bladder, it is able to support the gate in its retaining position.(Daniel & Paulus, 2018)

Below some characteristics of inflatable bladders are outlined:

- It takes relatively long to inflate and deflate with respect to other drive mechanisms. This naturally depends on the number and size of the bladders in the entire gate system. However, the system itself is less labor intensive in comparison with other gate drives.(Daniel & Paulus, 2018)
- Bladders should be anchored to the sill structure, which makes it difficult to inspect and perform maintenance. However, the rubber material does not experience corrosion which means that the bladder itself is low in maintenance. (Daniel & Paulus, 2018)
- The service life of bladder is generally around 30 years which is lower than other drives. Exposure to sunlight leads to a quickened aging process which reduces the service life. (Daniel & Paulus, 2018)
- Deflated bladders can be stored more easily with respect to other drives such as hydraulic cylinders. Moreover, since the material is rubber, which is lightweight, means that it does not require heave foundations and takes up little space. However, when it is inflated, it takes up a relatively large space. (Daniel & Paulus, 2018)
- Inflatable bladders as a gate drive is relatively new on the market with respect to other gate drives. (Daniel & Paulus, 2018)

In Figure F.2 an example of a wire rope system is shown.

Figure F.2: Example of gate system with an inflatable bladder (Daniel & Paulus, 2018)

G

Selection of barrier type

This Appendix verifies potential barrier types to a set of functional requirements and evaluates the remaining barrier types on several criteria to further narrow down the options to find a potential solution for a self-closing flood barrier.

G.1. Verification of barrier concepts

This appendix shows firstly a verification on the functional requirements of different concepts for the barrier type of the self-closing flood barrier. If a concept meets the requirement, it is indicated with a green tick in each table and otherwise with a red cross. If a concept could potentially meet the requirement, but is yet to be designed with respect to the requirement a blue circle is added. If there is no information available about a gate type with respect to a certain criterion, 'N/A' is added , which indicates that a verification for this gate type about this criterion is not available. From this verification table, it follows that the parachute barrier, the bellows barrier and the visor gate will not comply to the requirements in advance.
Explanatory notes

The visor gate does not comply to requirements:

- RQ-007: "Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes."
- RQ-008: "Structure has sufficient water retaining height derived from RQ-001"

The visor gate requires to have piers above ground level in order to ensure the rotational movement around the horizontal axis. The structure does have the ability to be embedded in the subsoil at an angle to the horizontal. However, in retaining position at the supports (piers), the gate would not have enough retaining height, because of the orientation of the gate and the rotational movement.

The bellows barrier does not comply to requirements:

- RQ-003: "Structure should be integrable in the area in existing structures, the subsoil or existing grass dikes."
- RQ-007: "Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes."

The bellows barrier is as of yet not executed with a continuous span of 240 m. The possibility of such an application is still uncertain and might be infeasible. This implicates to look at already proven feasible solutions for this gate type which would be an application with multiple spans with bellows. However this results in the requirement to have intermediate support structures above ground level which is not in compliance with the requirements.

The parachute barrier does not comply to requirement:

- RQ-007: "Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes."

The parachute barrier requires pylons or similar structures to clamp the top of the fabric with ropes in order to stretch it so that sufficient tensile stresses develop. For this purpose, the pylons need to be above the ground surface level which does not comply to the requirements.

Conclusion of the verification
The visor gate, bellows barrier and a parachute appear not to be in compliance for a self-closing flood barrier in this case. This is shown in the verification table in Figure G.1.

		Gate types						
	Functional requirements	**Flat**	**Flap**	**Sector**	**Radial**	**Visor**	**Bellows**	**Parachute**
RQ-001	Structure should be designed with an annual flood probability of 1:100 for the dike segment which is translated to stricter requirements per dike section per failure mechanism	o	o	o	o	o	o	o
RQ-002	Structure has a functionality lifetime of 40 years.	o	o	o	o	o	o	N/A
RQ-003	Structure should be integrable in the area in existing structures, the subsoil or existing grass dikes.	o	o	o	o	o	×	o
RQ-004	Structure closes itself both manually activated and autonomously.	o	o	o	o	o	o	o
RQ-005	Structure has adaptability for the water retaining function.	o	o	o	o	o	o	o
RQ-006	Structure offers possibility to test, inspect and maintain the system.	o	o	o	o	o	o	o
RQ-007	Structure has no townscape (including sight on the Meuse) obstructing elements if it is not in function, so should be submerged in the soil or embedded in existing structures or grass dikes.	o	o	o	o	×	×	×
RQ-008	Structure has sufficient water retaining height	o	o	o	o	×	o	N/A
RQ-009	Structure has a reliable closing system	o	o	o	o	o	o	N/A

Figure G.1: Verification of concepts of retaining wall against requirements

G.2. Evaluation of remaining barrier concepts

Below the explanatory notes are found that justify the scoring for each gate type per criterion. The underlying information can be found in Appendix F.

[1]: For a gate leaf with a curved shape with a radius, such as a sector gate or a radial gate, an adaptation to the retaining height requires more material and effort than for a flap or flat gate, since the gate extension is not purely in the vertical direction. This means more material is needed to gain the same

	Adaptibility	Integrability	Efficiency	Maintainability	**Total**	**Weight**
Adaptibility	1	1	1	1	**4**	**0.36**
Integrability	1	1	1	1	**4**	**0.36**
Efficiency	0	0	1	1	**2**	**0.18**
Maintenance	0	0	0	1	**1**	**0.09**

Figure G.2: Matrix to determine weights per criterion

		Sector	Radial	Flap	Flat	
Adaptibility	*0.36*	**1.0**	**1.0**	**3.0**	**4.0**	**[1]**
Efficiency	*0.18*	**2.5**	**2.0**	**3.5**	**3.0**	**[2]**
Functional simplicity		*2*	*1*	*4*	*4*	
Material usage		*3*	*3*	*3*	*2*	
Maintenance	*0.09*	**1.0**	**3.0**	**2.5**	**2.5**	**[3]**
Access		*1*	*3*	*4*	*1*	
Number of maintainable parts		*1*	*3*	*1*	*4*	
Integrability	*0.36*	**2.0**	**2.0**	**3.0**	**4.0**	**[4]**
Connection with area and surroundings		*1*	*1*	*3*	*4*	
Space usage		*3*	*3*	*3*	*4*	
Total		**1.6**	**1.7**	**3.0**	**3.6**	

Figure G.3: Trade-off matrix for multi-criteria analysis

heightening. A flat gate and flap gate both have a relatively simple execution with respect to adaptation in terms of the retaining height. The reason for this is that the gate leaf can be extended by simply welding an additional straight plated element to the gate leaf. For the flap gate this results in the need for additional reserved space across the width, as an obstacle free zone. For the flat gate this results in the need for additional reserved space in the height of the structure. The latter is considered less of a challenge than reserving more space for the obstacle free zone as is the case for a flap gate. This is in view of the spatial quality and the fact that it is private territory.

In conclusion, the sector and radial gate are scored with the lowest adaptability, mainly because of the curved skin plate and corresponding aspects as described in the above section. Then follows the flap gate with moderate adaptability, because of the convenience of the adapting to a straight skin plate yet with also the side note that the obstacle free zone of the gate increases in width, so the final grading is a ‘3’ instead of ‘2’. The flat gate is scored a little better than the flap gate with a moderate to high adaptability, because it is the easiest with respect to the other types, but of course it does require attention with respect to additional excavation and higher loads because of a lower foundation level, allowing the gate type to be scored with a ‘4’ instead of a maximum score of ‘5’.

[2]: The material usage for the radial gate and flap gate are in practice generally the lowest with re-

spect to the other gate types, based on the report Multifunctional movable flood barriers from Royal HaskoningDHV from Dijk and van der Ziel (2010) which elaborates on a similar trade-off as this report but for dams and weirs in rivers. This document evaluates among others the same gate types as potential solutions for dams and weirs. However, it still provides a sense to initially comment on the material usage. The reason for this is, is because each gate type has its own characteristics that are the same in every design problem, which differentiates them from each other. For example, the radial gate transfers loads via compression in the gate leaf eventually to compressive strut arms, leading to zero moments caused, whereas a flat gate uses global bending to transfer the forces eventually to the concrete foundation. Furthermore a flat gate uses both in-plane and out-of-plane stress distribution to transfer the forces to the concrete foundation. These characteristics are reflected in the use of materials, because the more bending is involved the more material is required because a heavier overall composed cross-sectional profile is needed for the gate leaf.

Furthermore, the gate types differentiate in the functional simplicity. Firstly, the flat gate has the advantage with respect to the functional simplicity over the other three gate types that the functionality is based on a linear motion, whereas the other three gate types are involved with rotational movements that require more movable parts. Furthermore, the flat gate entails a small obstacle-free zone in comparison to the others.

[3]: For the gate maintenance, access is very important. The flat gate has limited access, because of the narrow gate recess in the concrete foundation. This means that the gate leaf could be inspected and maintained primarily in its closed state. However, seals and support elements such as guides are still very difficult to access in the recess. For this reason the score regarding maintenance access is the lowest. The sector gate is also maintainable primarily in its closed state. To raise a sector gate which is characteristically driven by buoyancy, external hoist equipment is necessary like lifting trucks, as inspection and maintenance in its open state is poorly accessible (Dijk & van der Ziel, 2010). For this reason, the sector gate is also scored with ‘1’. Considering the radial gate, inspection to the gate and the trunnions are better accessible than the two last mentioned gate types, because the gate leaf is lowered in a recess in the concrete foundation where there is sufficient space because of the already required space to house the gate with strut arms regularly. Lastly, the flap gate will be considered. The flap gate is close to the surface level, which makes it easy to access. However, the hinged connections over the entire span are still not perfectly accessible (Daniel & Paulus, 2018), which makes that the flap gate is still graded with a ‘4’.

The number of maintainable parts is the highest for the sector, flap gate and radial because they involve rotating components. For the sector gate and the flap gate the number of maintainable parts are often more than for a radial gate because these gate types require hinge supports over the entire span with a rotating pin, whereas the radial gate generally only has two trunnions per gate section. The flat gate requires maintenance to for example guides, rollers and/or slides but these are not located over the entire span (Daniel & Paulus, 2018). Because trunnions require more extensive inspection and maintenance, the maintenance works are more than in case of a flat gate (Daniel & Paulus, 2018). The reasoning above follows the trade-off in Multifunctional movable flood barriers from Dijk and van der Ziel (2010), from which the scoring is performed.

[4]: The sector, radial and flap gate require a space width as an obstacle free zone that is equal to the retaining height, and thus are scored with a ‘3’, which corresponds to moderate usage of space. When the gate uses more than 4 m it is scored with a ‘1’ and when the observable space usage in the gardens is less than 1 m, it is scored the highest with a ‘5’. This is the case for a flat gate. Considering the connection with the area and surrounding structures, the flat gate generally consists of a deep slender foundation which is in need of more consideration than shallow and wide foundations. However, a slender foundation is less likely to coincide with present underground structures and surroundings. A sector gate is generally massive and heavily founded over the entire span which is not preferable when it comes to constructing a flood barrier in gardens. For the radial gate, the trunnions are heavily loaded since it is the main component where all the loads are directed to, which requires a heavy foundation. Dividing the loads to the foundation over a longer span reduces the size of the foundation. For the sector gate and radial gate generally, the installation requires high accuracy, which takes more con-

struction time and hindrance which is not preferable for the people living at the site. Also, in retained position the radial gate takes up a few metres of width because of the strut arm, depending on the curvature of the gate leaf, in order to rotate from open state to closed state. This has an effect on the obstacle free zone of the gate and with respect to the placement of the trunnions, the anchoring is a few metres more inwards towards the residents or the gate leaf is located a few metres more towards the river side, which means that the ground level elevation variations could be too large, since the area is known for its steep foreland.

H

Conceptualising barrier types with drive mechanisms

In this appendix barrier types are combined with drive mechanisms to create potential concepts for a self-closing flood gate system. For this conceptualisation, drafts will be presented for each combination of a barrier type with a drive mechanism.

The barrier types involved in this are a flat gate and a flap gate. The flat gate is a vertical flood wall which moves in the vertical plane and the flap gate is a hinged flood wall which rotates in the vertical plane. The drive mechanisms involved in the conceptualisation are a wire rope system, a cylinder driven system, a buoyancy driven system and an inflatable system.

Important notes with respect to the conceptualisation for every combination of barrier with drive mechanism are:

- The assumed retaining height based on the preliminary hydraulic boundary conditions is 2.0 m. The actual retaining height is larger and is determined with Hydra-NL. However for the conceptualisation the exact retaining is not yet important since the goal is to compare the different concepts to make a selection of the one with the most potential.
- The underground structure is made of concrete
- The barrier itself is made of steel.
- The dimensions are indicatively determined based on the retaining height and characteristics of the drive mechanisms. There are no structural calculations underlying this yet. They are solely indicated for comparison purposes.

H.1. Flat gate driven by cylinders

In the Figures H.1 and H.2 the gate concept is shown where Figure H.1 shows the position in rest and Figure H.2 shows the retaining position. Both figures show a cross-section and a longitudinal section view.

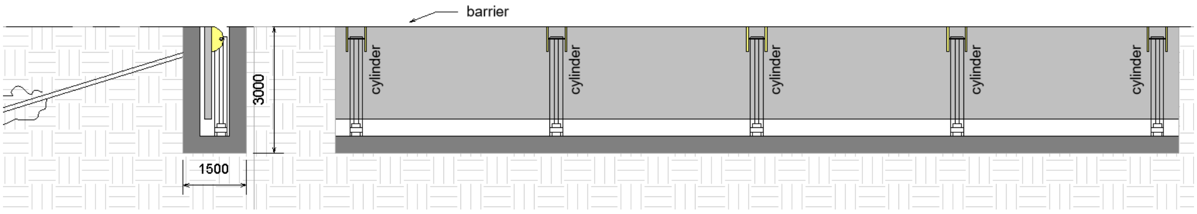


Figure H.1: Draft of flat gate driven by cylinders in resting position. Left: cross-section; Right: longitudinal section

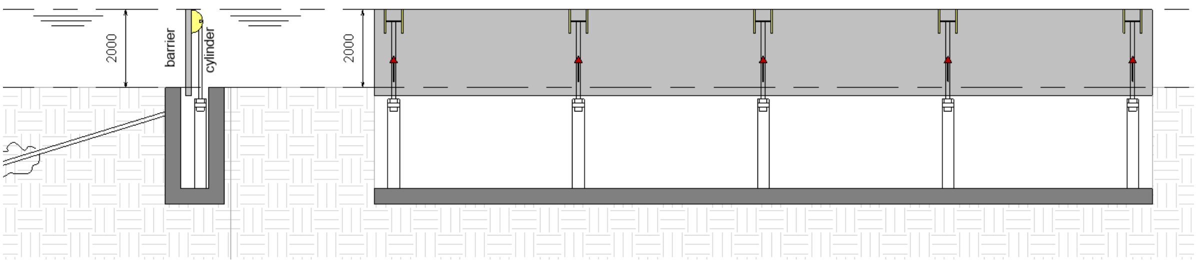


Figure H.2: Draft of flat gate driven by cylinders in retaining position. Left: cross-section; Right: longitudinal section

Important characteristics for this concept:

- The structure is quite small in overall width. This has an advantage with respect to the construction hindrance and the amount of excavation. However, a small width may lead to insufficient rotational stability due to the moment exerted by the hydraulic loads. For this reason, additional measures may be required such as a permanent anchor (see Figures H.1 and H.2) combined with sheetpile walls, a piled foundation or for example a gravity structure.
- For the cylinders, power units are required that need to be stored in machine rooms
- The hydraulic load on the gate will exert a moment on the barrier. This moment needs to be transferred to the concrete underground structure. Since the gate is kind of suspended from the cylinders, the cylinders are also subjected to this moment. However cylinders have a different purpose. Thus the gate requires sufficient bending stiffness and a console at the top of the concrete structure that allows the gate to transfer the loads to the concrete without loading the cylinders in transversal direction.

H.2. Flat gate driven by a wire rope system

In the Figures H.3, H.4 and H.5 the gate concept is shown where Figure H.3 shows the position in rest and Figure H.4 shows the retaining position. Both figures show a cross-section and a longitudinal section view. Figure H.5 shows a top view.

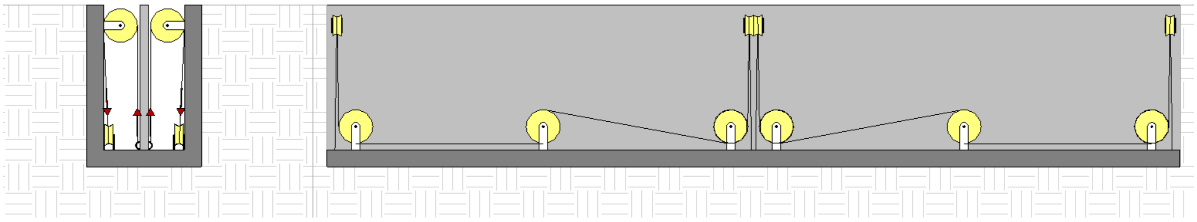


Figure H.3: Draft of flat gate driven by a wire rope system in resting position. Left: cross-section; Right: longitudinal section

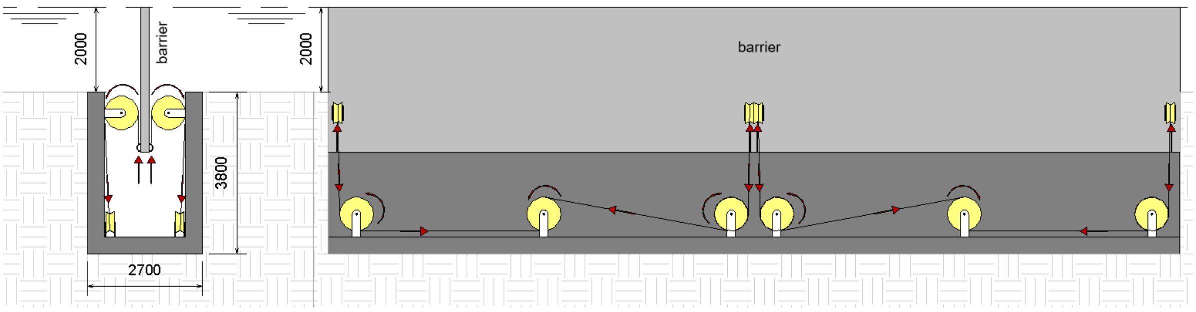


Figure H.4: Draft of flat gate driven by a wire rope system in retaining position. Left: cross-section; Right: longitudinal section

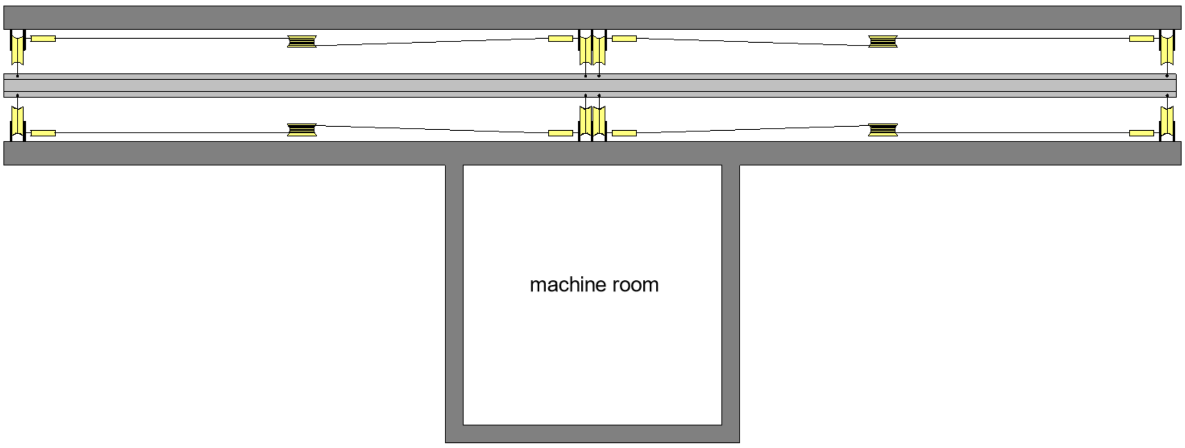


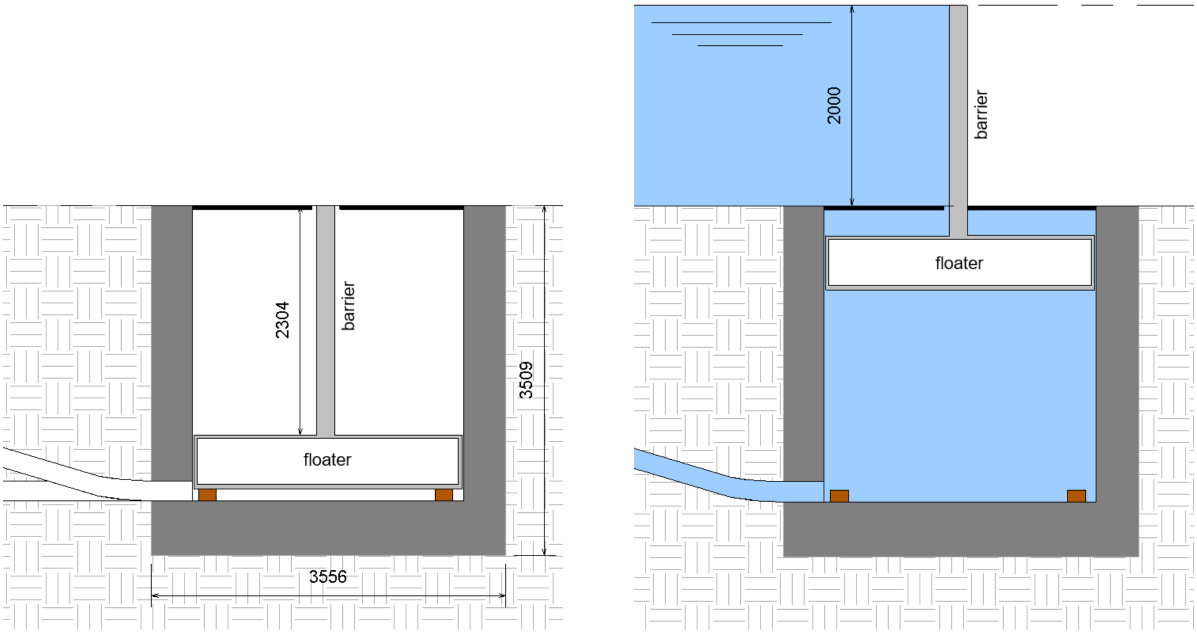
Figure H.5: Draft of a top view of a flat gate driven by a wire rope system

Important characteristics for this concept:

- The structure is quite small in overall width like with the cylinder driven flat gate. This has an advantage with respect to the construction hindrance and the amount of excavation. However, a small width may lead to insufficient rotational stability due to the moment exerted by the hydraulic loads. For this reason, additional measures may be required such as a permanent anchor combined with sheetpile walls, a piled foundation or for example a gravity structure.
- For the cable drums to start rotating and winching, power units are required that need to be stored in machine rooms.
- The hydraulic load on the gate will exert a moment on the barrier. This moment needs to be transferred to the concrete underground structure. Since the gate is kind of suspended from the wire ropes, the gate is sensitive to tilting. Thus the gate requires a sufficient bending stiffness and a console within the concrete structure that allows the gate to transfer the loads to the concrete. Also lateral guidance is important to prevent tilting.
- As is indicated in the Figures H.3, H.4 and H.5 the system needs quite a few cable drums for a span of 20 m.

H.3. Flat gate driven by buoyancy

In the Figures H.6a and H.6b the gate concept is shown where Figure H.6a shows the position in rest and Figure H.6b shows the retaining position. Both figures show a cross-section.



(a) Draft of flat gate driven by buoyancy in resting position

(b) Draft of flat gate driven by buoyancy in retaining position

Figure H.6: Draft of flat gate friven by buoyancy

Important characteristics for this concept:

- The inside of the concrete structure functions as the basin for the water storage to create a water body in which the barrier stays afloat.
- The floater needs sufficient volume to displace a water volume to induce the required buoyancy
- The system needs a water supply network with for example pipes
- The hydraulic load on the gate will exert a moment on the barrier. This moment needs to be transferred to the concrete underground structure. The connection between the floater and the concrete will provide this.
- Lateral guidance is important to prevent tilting.

H.4. Flat gate driven by inflation

In the Figures H.7, H.8 and H.9 the gate concept is shown where Figure H.7 shows a cross-section of the position in rest and Figure H.8 shows a cross-section of the retaining position. Figure H.9 shows a cross-section where no machine room is present, which indicates a small overall width of the structure.

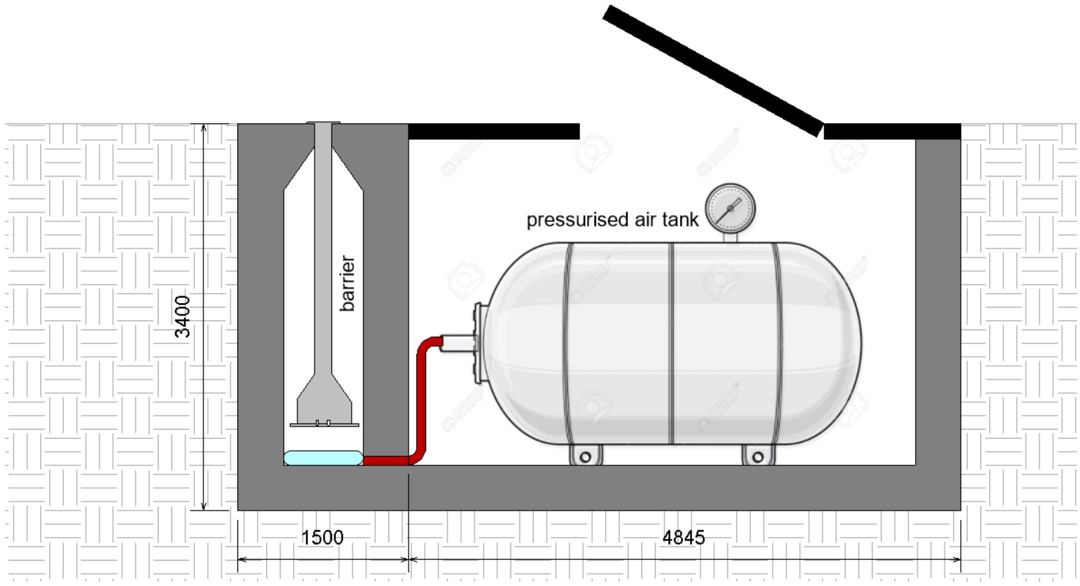


Figure H.7: Draft of flat gate driven by inflation in resting position.

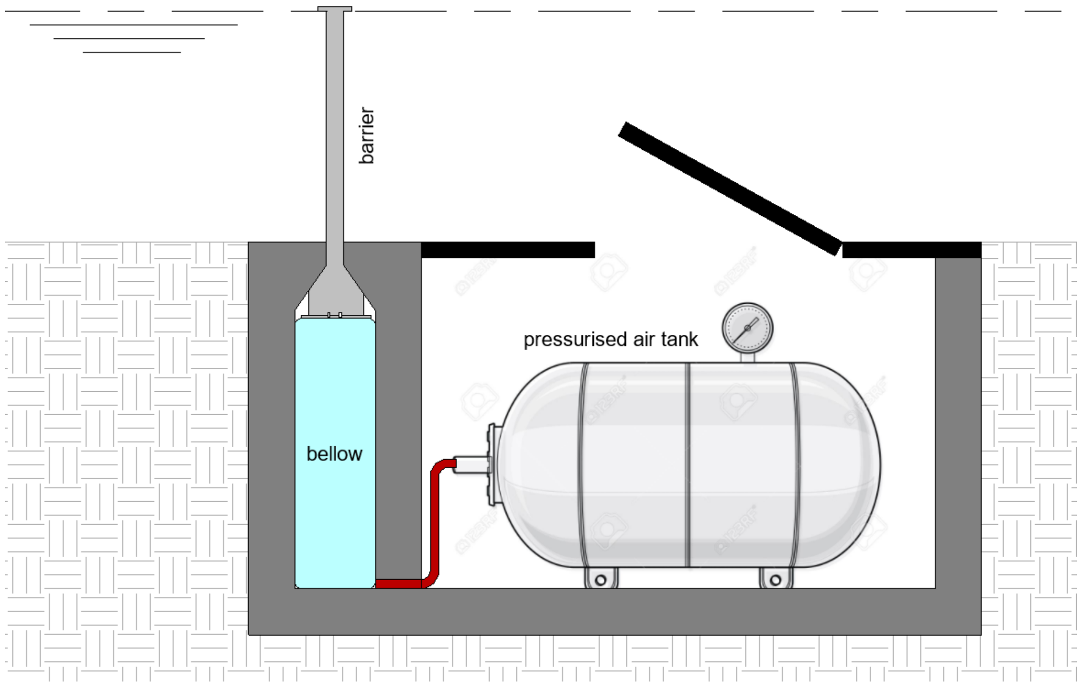


Figure H.8: Draft of flat gate driven by inflation in retaining position.

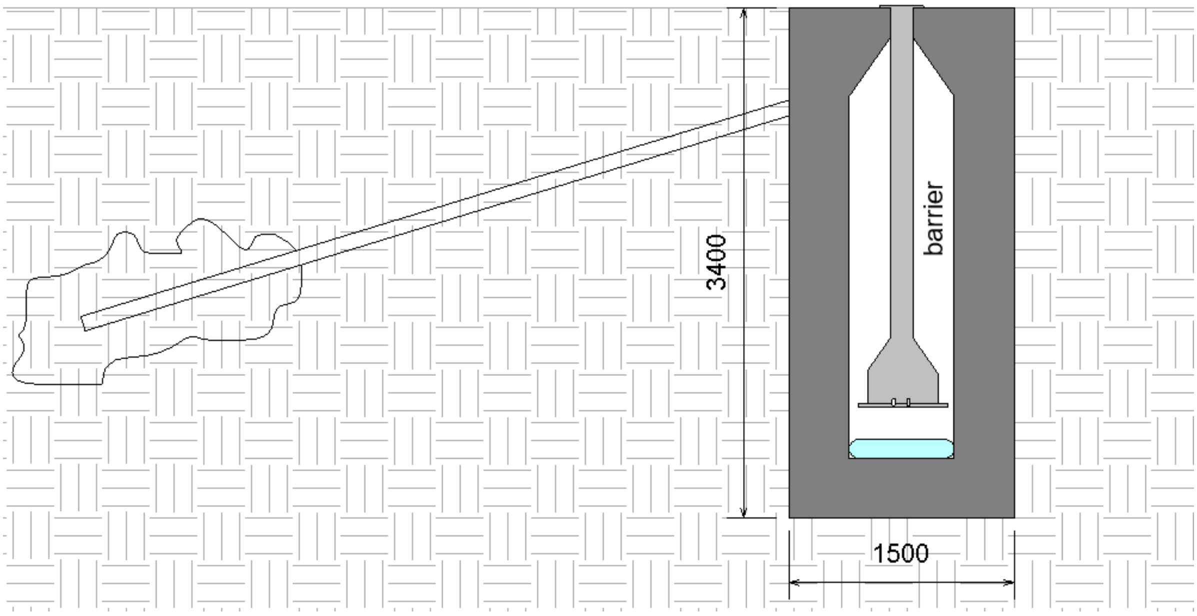


Figure H.9: Cross-section of gate without machine room

Important characteristics for this concept:

- The gate system required pressure vessels for the air supply. The bellows requires 100 to 200 kN/m² overpressure to be inflated and provide sufficient pressure (Rijkswaterstaat, 2005). Pressure vessels are available with a supply of a pressure of 1100 kN/m² (Airpress, 2023) which means that a particular system is feasible and it would require several pressure vessels along the span.
- For the pressure vessel to supply air into the bellows, a machine room is required to store the required appliances per section.
- Where the structure does not have a machine room adjacent to it, the overall width is quite small. This has an advantage with respect to the construction hindrance and the amount of excavation. However, a small width may lead to insufficient rotational stability due to the moment exerted by the hydraulic loads. For this reason, additional measures may be required such as a permanent anchor (see Figure H.9) combined with sheetpile walls, a piled foundation or for example a gravity structure.
- The inside of the concrete structure needs smooth surfaces in order for the bellows to inflate freely and not cause friction force on the concrete
- The material of the bellows need sufficient elasticity and special manufacturing to adjust to the inner shape of the concrete structure

H.5. Flap gate driven by cylinders

In Figure H.10 one gate concept is shown with the position in rest and in the retaining position. In Figure H.11 a second gate concept is shown with the position in rest and in the retaining position. The main difference between these two concepts is the closure of the gate by either a cylinder with traction or a cylinder with thrust.

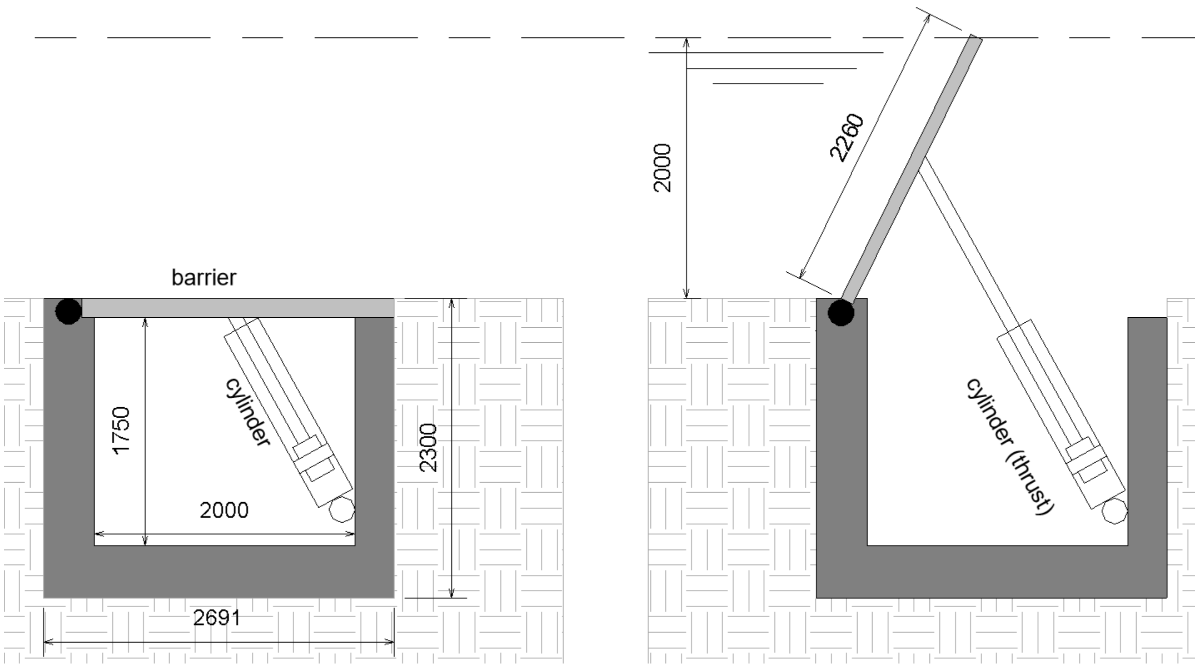


Figure H.10: Draft of one concept for flap gate driven by cylinders. Left: in resting position; Right: retaining position

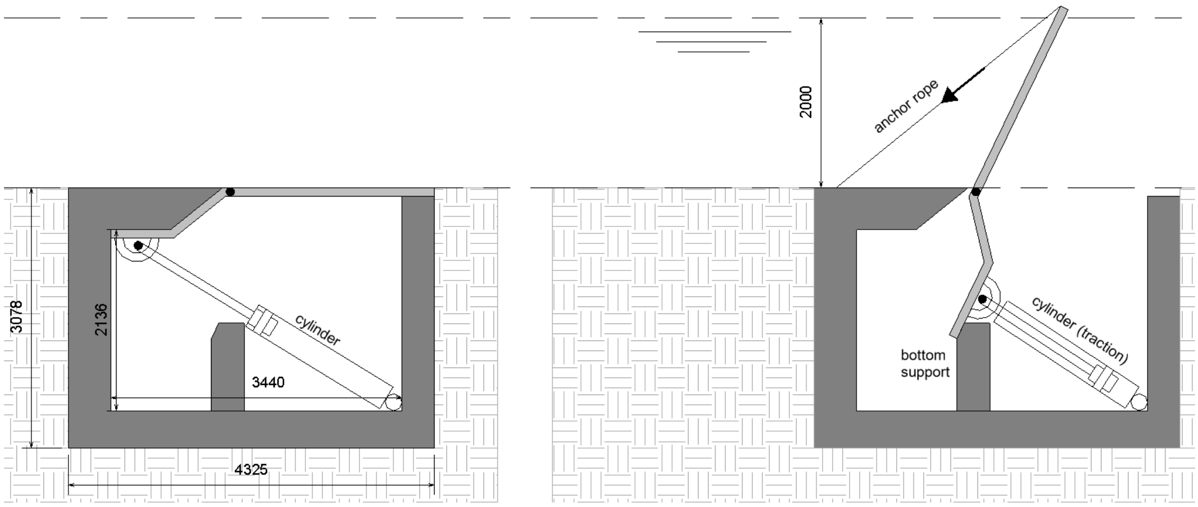


Figure H.11: Draft of second concept for flap gate driven by cylinders. Left: in resting position; Right: retaining position

Important characteristics for this concept:

- For the cylinders, power units are required that need to be stored in machine rooms
- For the first concept, shown in Figure H.10, the hydraulic load on the gate will partly be transferred to the cylinders. The cylinders need to be designed with this extra support (thrust) force. The loads are also transferred to the trunnion which directs them in turn to the concrete
- For the second concept, shown in Figure H.11 the gate is executed with an anchor rope to prevent flipping back in open position. These anchor ropes also provide top support. For the bottom support of the gate, a support block within the concrete structure is required to prevent additional rotation and to transfer the loads from the gate to the concrete without loading the cylinders.

H.6. Flap gate driven by a wire rope system

In Figure H.12 the gate concept is shown with the position in rest and in the retaining position.

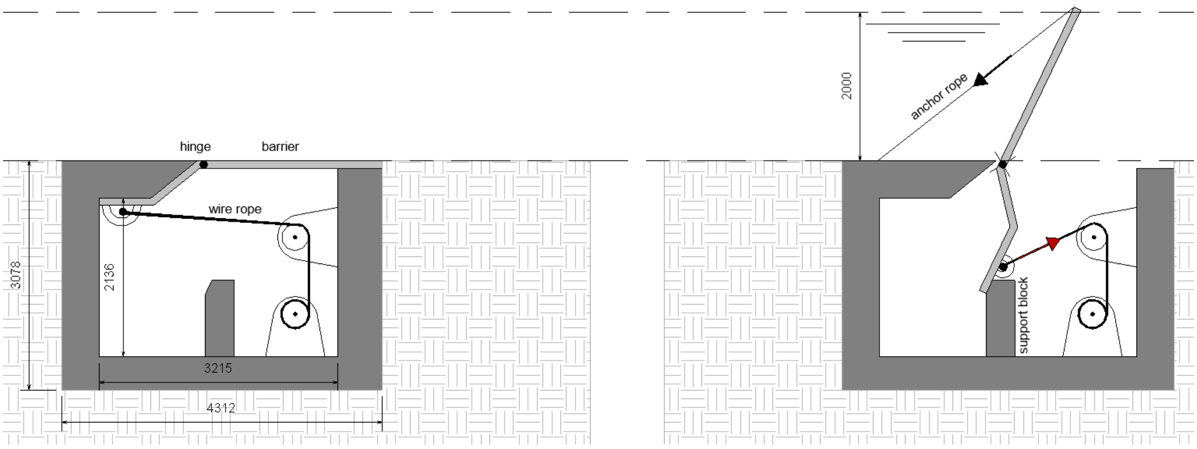


Figure H.12: Draft of flap gate driven by a wire rope system . Left: in resting position; Right: in the retaining position

Important characteristics for this concept:

- For the cable drums to start rotating and winching, power units are required that need to be stored in machine rooms.
- The gate is executed with an anchor rope to prevent flipping back in open position. These anchor ropes also provide top support. For the bottom support of the gate, a support block within the concrete structure is required to prevent additional rotation and to transfer the loads from the gate to the concrete
- The system needs quite a few cable drums for a span of 40 m.

H.7. Flap gate driven by buoyancy

In Figure H.13 and Figure H.14 the concept for a flap gate driven by buoyancy is shown. In Figure H.13 the concept is shown with one configuration and in Figure H.14 the concept is shown with a second configuration. The main difference between these two figures is the rotational direction of the closure of the gate, which influences the static equilibrium of the forces and therefore the force in the anchor ropes. Furthermore, it affects whether the barrier is directly loaded by the hydraulic loads or indirectly.

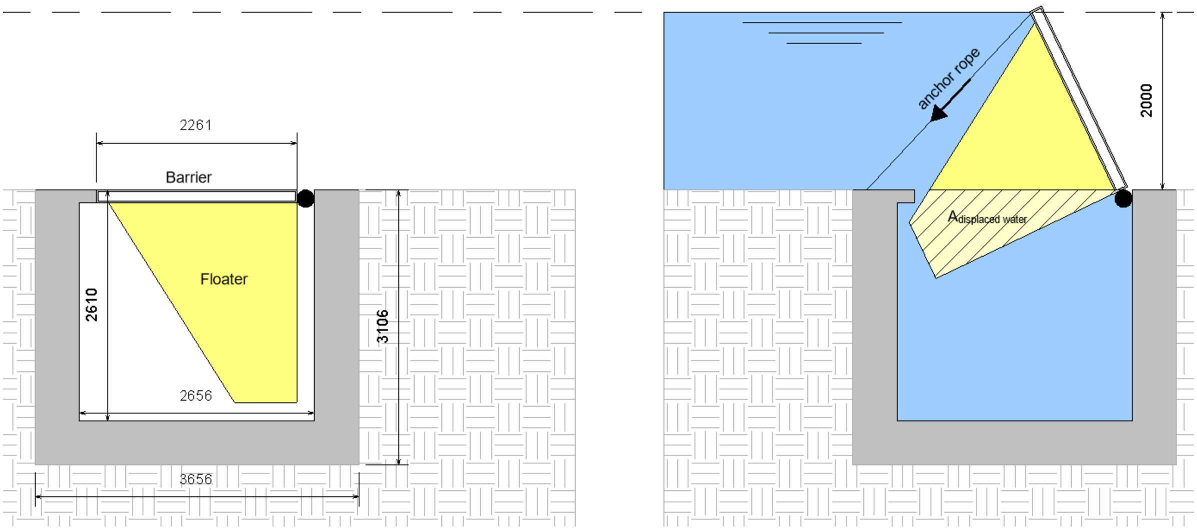


Figure H.13: Draft of concept for flap gate driven by buoyancy with clockwise rotation. Left: in resting position; Right: retaining position

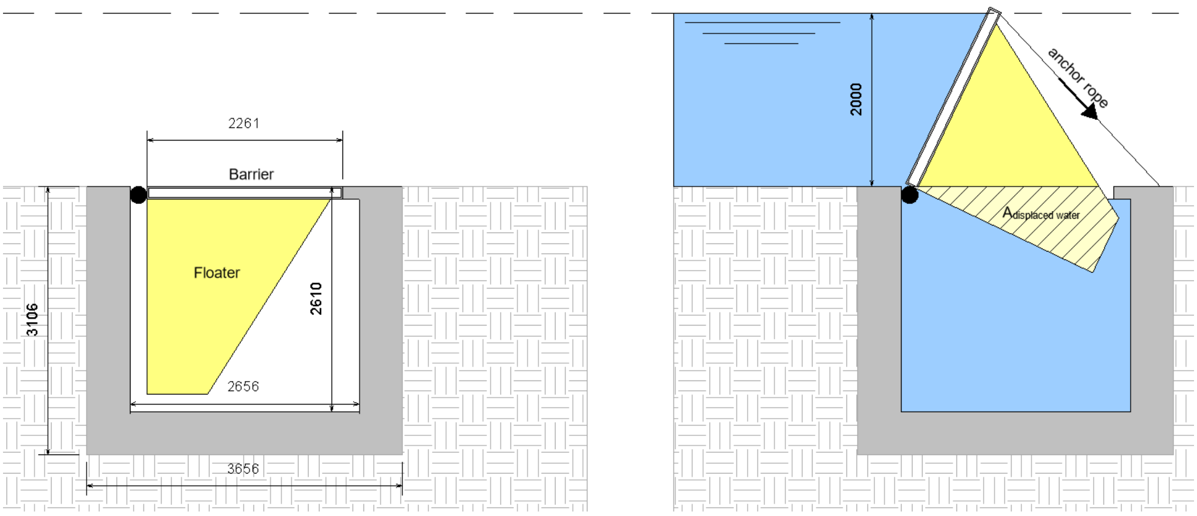


Figure H.14: Draft of concept for flap gate driven by buoyancy with anti-clockwise rotation. Left: in resting position; Right: retaining position

Important characteristics for this concept:

- The inside of the concrete concrete functions as the basin for the water storage to create a water body in which the barrier stays afloat.
- Anchor ropes are required to prevent flipping over by the hydraulic loads.

- The floater of the barrier requires certain dimensions in order to provide sufficient buoyancy by displacing a certain water volume
- The system needs a water supply network with for example pipes
- The loads are transferred to the trunnion which directs them in turn to the concrete

H.8. Flap gate driven by inflation

In the Figures H.15 and H.16 the gate concept is shown where Figure H.15 shows a cross-section of the position in rest and in retaining position and Figure H.16 shows a cross-section of the gate concept in retaining position where a machine room is present.

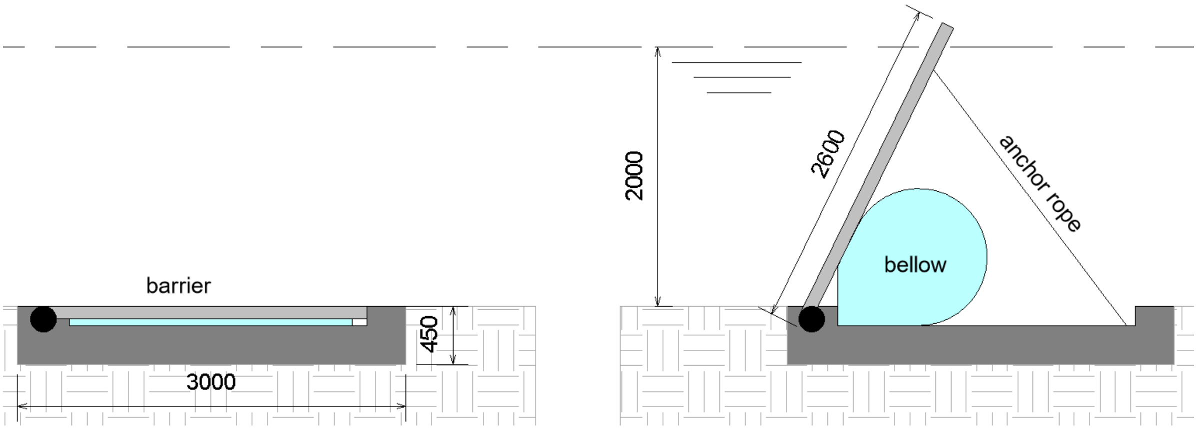


Figure H.15: Draft of flap gate driven by inflation

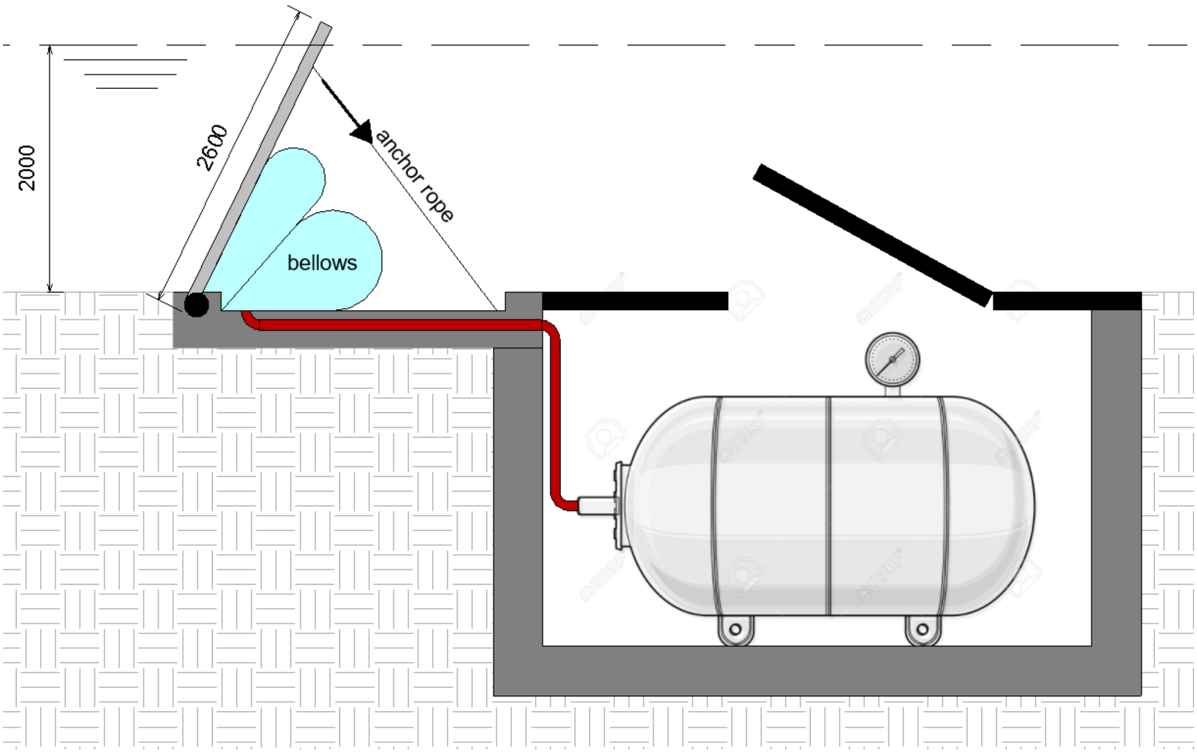


Figure H.16: Draft of flap gate driven by inflation in retaining position.

Important characteristics for this concept:

- The gate system required pressure vessels for the air supply. The bellows requires 100 to 200 kN/m² overpressure to be inflated and provide sufficient pressure (Rijkswaterstaat, 2005). Pressure vessels are available with a supply of a pressure of 1100 kN/m² (Airpress, 2023) which means that a particular system is feasible and it would require several pressure vessels along the span.

- For the pressure vessel to supply air into the bellows, a machine room is required to store the required appliances per section.
- The structure has a very small foundation depth in comparison to the other concepts
- The gate is anchored with a rope to prevent flipping over.

Selection of barrier type with drive mechanism

I.1. Multi-Criteria Analysis

In Figure I.2 the trade-off matrix is shown in which the gate types with driving mechanisms are graded based on the evaluation criteria from the basis of design. The justification for the scoring is given underneath the table. Also, the determination of the weights given to each criterion is shown in Figure I.1. If the criterion on the vertical axis is more important than the criterion on the upper horizontal axis, a '1' is given in the upper triangle of the matrix and a zero in the lower triangle in the diagonally mirrored cell. Each criterion receives a '1' with respect to its own. If the criteria are equally important, they both receive a '1'. In the end, for each criterion the total score is divided by the total points that are given to all criteria, which results in the weight factor.

	Reliability	Complexity	Maintenance	Sustainability	Integrability	Adaptibility	**Total**	**Weight**
Reliability	1	1	1	1	1	1	**6**	**0.23**
Complexity	0	1	1	1	1	1	**5**	**0.19**
Maintenance	0	1	1	1	1	0	**4**	**0.15**
Sustainability	0	0	0	1	0	0	**1**	**0.04**
Integrability	0	1	1	1	1	1	**5**	**0.19**
Adaptibility	0	1	1	1	1	1	**5**	**0.19**

Figure I.1: Matrix for determining weight factors per each evaluation criterion for the MCA

	Weight	Flat gate: Cylinder	Flat gate: Wire rope	Flat gate: Buoyancy	Flat gate: Inflation	Flap gate: Cylinder	Flap gate: Wire rope	Flap gate: Buoyancy	Flap gate: Inflation
Operational reliability	*0,23*	**3,5**	**3,0**	**4,5**	**2,0**	**3,5**	**3,0**	**4,5**	**2,0**
Back-up drive		*3*	*2*	*4*	*1*	*3*	*2*	*4*	*1*
Ease of operation		*4*	*4*	*5*	*3*	*4*	*4*	*5*	*3*
Complexity	*0,19*	**3,0**	**2,5**	**4,0**	**2,0**	**1,5**	**1,5**	**3,0**	**1,5**
Space usage		*4*	*4*	*3*	*2*	*2*	*2*	*2*	*1*
Ease of construction		*2*	*1*	*5*	*2*	*1*	*1*	*4*	*2*
Maintenance	*0,15*	**2,5**	**1,5**	**2,5**	**2,0**	**3,5**	**2,5**	**3,5**	**3,0**
Access for maintenance		*2*	*1*	*1*	*1*	*4*	*3*	*3*	*3*
Necessity of maintenance		*3*	*2*	*4*	*3*	*3*	*2*	*4*	*3*
Sustainability	*0,04*	**3,0**	**3,0**	**5,0**	**4,0**	**3,0**	**3,0**	**5,0**	**4,0**
Adaptibility	*0,19*	**4,0**	**4,0**	**4,0**	**4,0**	**3,0**	**3,0**	**3,0**	**3,0**
Integrability	*0,19*	**4,0**	**4,0**	**4,0**	**4,0**	**3,0**	**3,0**	**3,0**	**3,0**
Total		**3,4**	**3,1**	**3,9**	**2,8**	**2,9**	**2,6**	**3,5**	**2,5**

Figure I.2: Trade-off matrix for barrier concepts including the flat and flap gate with drive mechanisms

I.2. Explanatory notes

I.2.1. Operational reliability

The operational reliability is subdivided into two subcriteria, namely the operational ease and the way of recovering the closing mechanism through a back-up drive after failure of the main drive.

Back-up drive

The back-up drives per each closing mechanism does not differentiate between the flat and flap gate. However, it is different for each driving mechanism to close the gate alternatively in case of system failure. For buoyancy this is the easiest, because the system solely has to be filled with water in order for the gate to raise in the closed position. This could be done by a back-up system with a water truck to pump water into the floatation chamber from the surface. This does not require intensive labour, other than setting up the water pump. Also, by checking whether the floatation chamber contains water, failure of inundation can be observed in an early stage in order to recover the system. However, since this is still not a perfect alternative measure, it is not scored perfectly. Cylinders follow hereafter, because they can be replaced in case of direct failure of the cylinder, but such tasks require activities directly to the gate which may not be preferable in case of expected high water levels. If the failure is related to the power source, a back-up system could manage the system to work or the cylinder could be driven through a hand pump. Failure of wire ropes would lead to the need for an alternative drive where the gate is lifted externally or manually by a rotating handle. Again for the wire rope it holds that power source related failure could be solved by a back-up system. Buoyancy driven systems do not need an external power source. Inflatable systems are hard to alternatively activate. Failure will be shortage or failing of air supply, caused by air leakage somewhere in the system. Locating the leakage spot and recovering the system is not easy and this system would require an external lifting equipment to raise the barrier.

In Figure I.3 a point-by-point summary is given of the justification in terms of advantages and disadvantages that result in a positive score point '+1' respectively a negative score point '-1' on the total score given in the MCA.

	Advantages	Disadvantages
Cylinder	• Option to be replaced if damaged (+1)	• Replacement activities are not often preferable in case expected extreme water levels (-1) • Back-up power source required (-1) • Manual back-up drive by hand pump (-1)
Wire rope		• Manual back-up drive by a rotating handle (-1) • Back-up power source required (-1) • External lifting equipment required if drum experiences blockage (-1)
Buoyancy	• No back-up power source required (+1) • Failure of inundation can be observed early (+1) • No manual drive required with intensive labour, but only connecting a water pump (+1)	
Inflation		• Locating leaking spot and recovery for inflating is hard (-2) • No manual drive possible other than with external lifting equipment (-1) • Back-up power source required (-1)

Figure I.3: List of advantages and disadvantages of the drive mechanisms regarding the back-up drive

In conclusion, buoyancy is the easiest and most simple to alternatively activate the system other than its original means. After this, cylinder driven systems follow, because they do have the option to be manually driven or to be replaced before external measures need to be taken. For wire ropes, this is not the case if the drums experience blockage. However, they can be manually driven if the failure is caused by power outage. Inflation is the least favorable with respect to an alternative drive.

Ease of operation

The driving mechanism involving buoyancy is the most easy in operating, because it does not involve switching on a motor or hydro power unit, such as with the driving mechanisms involving cylinders, wire rope and inflation. A buoyancy driven system solely needs water supply inundating the floatation chamber making the gate raise to its retaining position. The driving mechanisms including a hydro power unit require also a sensor in order to allow the system to work autonomous, i.e. ensuring the self-closing characteristic, meaning that there is yet another additional process in the entire operation, where a system driven by buoyancy does not require it, because the self-closing characteristic is already part of the closing mechanism.

Considering stopping of the movement of the gate, the driving mechanisms that use a hydro power unit and/or electric motor need a brake or limit switch to ensure the end of the motion of the gate. For a drive mechanism involving only buoyancy, this is not necessary, because the gate floats along with the water level in the floatation chamber, which in turn follows the water level in the Meuse. The gate ends automatically provided that there is a locking feature at the top of the floatation chamber in order to prevent the gate from floating out of the chamber. With mechanically driven systems, if the movement is not prevented at its predetermined limit, the embedded structure will be needlessly subjected to the maximum driving force, which is highly undesirable. In conclusion, the driving mechanisms including a hydro power unit or electric motor need an additional stopping process, which reduces the simplicity of the closing operation.

For a flat gate with an inflatable drive mechanism , that even adds to the fact that, deflation of the bellows proceeds in an uncontrolled way leading to the possibility that the gate leaf subsides in a tilted position. This could cause additional deformations and stresses in the gate leaf. This means that clamping lines should be added to the inner sides of the wall of the concrete chamber in order to deflate the bellows in a controlled way. This reduces the ease of the operation.

In Figure I.4 the different operational processes are summarised in process trees. It can clearly be seen that a buoyancy driven system is the most easy in operating.

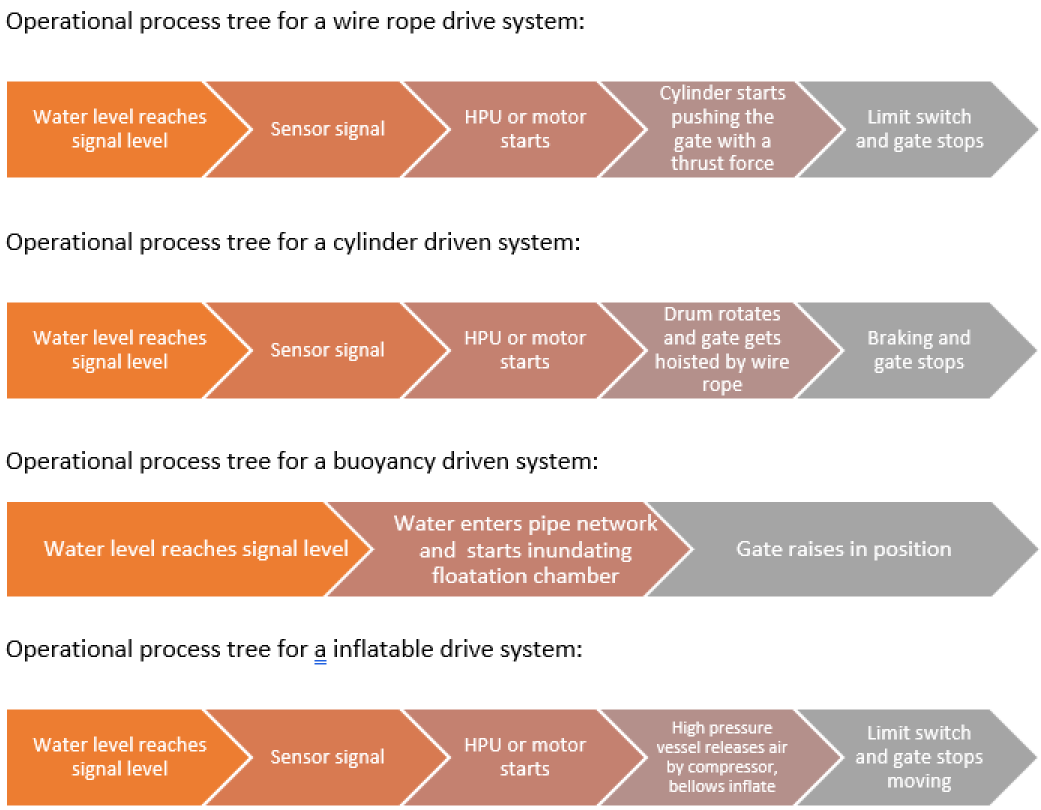


Figure I.4: Operational process trees for each drive mechanism

I.2.2. Complexity

The complexity is subdivided into two subcriteria, namely the construction ease and the amount of space usage each barrier type with drive mechanism requires.

Space usage

Floating gate systems requires per sub-span a certain pipe network to fill up the system. For this case the amount of space required for the pipes is estimated on 30m² with 340m of total pipe length for the entire dike section. It does have the advantage that no space is needed for locating a hydro power unit for example or other mechanical devices necessary. The approximate width for the floatation chamber of a flat gate is around 2 m in order to produce sufficient buoyancy, which leads to an approximate total area usage of 490 m². For this reason it is scored with a satisfactory performance (400 m²– 600 m²). For a cylinder driven system, every flat gate section of 30 m needs a minimum of two cylinders which take up an estimated width of 0.8 m. Every gate section needs a hydro power unit at the end of a gate section, so in total nine hydro power units are required. A hydro power unit is stored in a machine room together with motors, hoses, pipes and oil tanks. The approximate area for such a machine room is estimated at 20 m². A cylinder driven system thus requires a total space usage of 365 m², which is graded with good performance for space usage (200 m² - 400 m²). For a wire rope gate system this is estimated to be the same. For an inflating mechanism the width is estimated to be the same as a floating gate, so approximately 2 m. Because this mechanism requires pressure vessels, also machine rooms with electric motors are necessary. This leads to a total space usage of approximately 640 m² which is graded with sufficient performance (600 m² - 800 m²). This is similar for the flap gate driven by cylinders and wire ropes. A flap gate with buoyancy tanks or a floater has approximately a width around 2.5 m and a pipe network which leads to 605 m² of total space usage. And for the flap gate with inflating mechanisms it is estimated to have a space usage of 755 m², considering the same weight as the flat gate width with a buoyancy mechanism.

In Figure I.5 the estimated space usage for each concept is summarised with the associcated score in the MCA.

Required parts or span	**Area/Length**	**Unit**
9 machine rooms	180	m^2
Pipe network	30	m^2
Span dike section	230	m

Gate system	Estimated gate width [m]	***Area gate [m²]***	Machine rooms	Pipe network	***Additional required area [m²]***	**Total Area [m²]**	*Score*
Flat gate + cylinder	0.8	***185***	Yes	No	***180***	**365**	*4*
Flat gate + wire ropes	0.8	***185***	Yes	No	***180***	**365**	*4*
Flat gate + buoyancy	2.0	***460***	No	Yes	***30***	**490**	*3*
Flat gate + inflation	2.0	***460***	Yes	No	***180***	**640**	*2*
Flap gate + cylinder	2.0	***460***	Yes	No	***180***	**640**	*2*
Flap gate + wire ropes	2.0	***460***	Yes	No	***180***	**640**	*2*
Flap gate + buoyancy	2.5	***575***	No	Yes	***30***	**605**	*2*
Flap gate + inflation	2.5	***575***	Yes	No	***180***	**755**	*1*

Figure I.5: Estimated space usage per concept

Ease of construction
Regarding the self-closing characteristic, only buoyancy driven mechanisms do not require a sensor to have an autonomous activation. The system is already self-closing, which is an advantage over the others. The flap gate needs lateral supports between the hinge supports in order for the gate to behave statically and not dynamically. The reason for this is because it is bottom hinged supported but is free to displace between the hinge supports, potentially causing overrotation for example. This requires anchor ropes along the span preventing the gate to move in the other direction, which requires more attention to the design of the gate system.

The amount of mechanical parts is the least with a buoyancy driven system. Thereafter, an inflatable system follows. The wire rope has the most mechanical components, because it needs several drums about which the wire ropes are winched. The drum rotates, which means it has gears and/or brakes to control the rotational movement. The connection of the wire ropes with the gates need mounted hoist eyes. The wire rope drums require also connection and synchronization between them through a line shaft or a torque tube, to raise the gate synchronized. The cylinder performs sufficiently in the MCA with respect to the amount of mechanical parts. It has certain less mechanical components than a wire rope mechanisms but expected to have more than an inflatable system.

Lastly, the manufacturing of bellows for a flat gate needs special attention, because of the enclosed chamber with a specific shape, the bellows are limited in their shape and may need a special design with for example clamping lines, which will require extra research and specific manufacturing which will increase the complexity.

Note: the grading is done as such that in the case there is a clear advantage with respect to other variants, there is given a ‘1’, when there is a clear disadvantage with respect to other variants there is given a ‘-1’, and for the ‘mechanical parts’ the order for few to many parts is used to determine the score with respectively 5 to 1. In the last column of the table a corrected score is given with respect to the one but last column, because in the actual MCA-table only scores from 1 to 5 are used, meaning that a final score below ‘1’ in the one but last column, will be graded with ‘1’ in the last column and a score higher than ‘5’ will end up with a ‘5’.

In Figure I.6 the scoring for each concept is summarised.

	Autonomous activation	Stability raised position	Mechanical parts	Manufacturing	**Total score**	***Total score corrected***
Flat gate + cylinder	0	0	2	0	**2**	***2***
Flat gate + wire ropes	0	0	1	0	**1**	***1***
Flat gate + buoyancy	1	0	4	0	**5**	***5***
Flat gate + inflation	0	0	3	-1	**2**	***2***
Flap gate + cylinder	0	-1	2	0	**1**	***1***
Flap gate + wire ropes	0	-1	1	0	**0**	***1***
Flap gate + buoyancy	1	-1	4	0	**4**	***4***
Flap gate + inflation	0	-1	3	0	**2**	***2***

Figure I.6: Scoring for the construction based on autonomy, number of (additional) parts and manufacturing

I.2.3. Maintenance
The maintenance aspect is subdivided into two subcriteria, namely the access to the components responsible for the drive mechanism to perform maintenance and the necessity to perform maintenance.

Access for maintenance
It is clear that a flat gate has less access than a flap gate, because of the smaller width and larger depth of the concrete chamber with respect to a flap gate. For this reason the flat gate is scored lower with respect to this criterion for all driving mechanisms. Differentiation between the driving mechanisms

lead to the observation that cylinders are the most accessible with respect to the others. Clogged pipes in a large network for a buoyancy driven system are hard to directly access. Besides, accessing wire ropes under tension that are winched about a drum are also not easy to access for maintenance or inspection. The same holds for a deflated bellows below the gate leaf. For this reason, the access for maintenance for the latter three mechanisms for a flat gate is graded poorly. For a flap gate with the mechanisms involving wire rope, buoyancy and inflation, it is graded satisfactory, because the accessibility is in turn better than for a flat gate with cylinder, which is graded sufficiently. Lastly the flap gate with cylinder is graded good, because it is best accessible with respect to the others.

In Figure I.7 the above mentioned advantages and disadvantages are summarised with the associated score points assigned. All concepts start with a satisfactory grade of ’3’ and end up with a score in the MCA table by summing up the negative and positive score points to the starting value.

	Advantages	Disadvantages
Flat gate + Cylinder	• Cylinders normally accessible inspection, maintenance and repair (+0)	• Small gate width for access (-1) • Cylinders hard to access for inspection, maintenance and repair
Flat gate + Wire rope	•	• Small width for access (-1) • Winched part of wire ropes are hard to inspect, winching needed and intensive work to disassemble (-1)
Flat gate + Buoyancy	•	• Small width for access (-1) • Pipes are highly inaccessible, excavation necessary (-1)
Flat gate + Inflation	•	• Small width for access (-1) • Bellows are highly inaccessible, gate needs to be lifted from embedded concrete chamber (-1)
Flap gate + Cylinder	• Gate recess good accessible (+1)	•
Flap gate + Wire rope	• Gate recess good accessible (+1)	• Winched part of wire ropes are hard to inspect, winching needed, and intensive work to disassemble (-1)
Flap gate + Buoyancy	• Gate recess good accessible (+1)	• Pipes are highly inaccessible, excavation necessary (-1)
Flap gate + Inflation	• Gate recess good accessible (+1)	• Bellows are highly inaccessible for inspection, gate need to be lifted (-1)

Figure I.7: List of advantages and disadvantages of the drive mechanisms regarding the accessibility for performing maintenance

Necessity for maintenance

The gate types with driving mechanisms do not differentiate in the necessity of maintenance. Important to note is that no mechanisms are assumed to be completely maintenance free, i.e. leading to a score of ’5’. However, there remain differences between the drive mechanisms. A buoyancy driven system requires little maintenance, because it has no mechanical components, but do need yearly unclogging or cleaning of the pipe network from debris that may have entered the system, so this mechanism is graded with a good performance regarding the necessity for maintenance. Cylinders and inflatable bellows come next in this listing, because they require little maintenance as well but do have mechanical parts and will thus be scored with a satisfactory performance regarding the necessity to maintenance. Wire rope mechanisms by contrast have the most mechanical components and thus require also the most maintenance relative to the beforementioned ones. Because the frequency of the maintenance is not clearly quantifiable, it is still scored with sufficient performance.

In Figure I.8 the above mentioned advantages and disadvantages are summarised with the associated score points assigned. All concepts start with a good grade of ’4’ with respect to the necessity of maintenance and end up with a score in the MCA table by summing up the negative and positive score points to the starting value. Little maintenance is advantageous but is scored neutrally since the starting grade is already ’4’.

	Advantages	Disadvantages
Cylinder	• Little maintenance required (+0)	• Mechanical components (-1)
Wire rope		• Mechanical components (-1) • Rotational components (-1)
Buoyancy	• No mechanical components that require maintenance (+1)	• Yearly unclogging and cleaning of pipe network required (-1)
Inflation	• Little maintenance required (+0)	• Mechanical components (-1)

Figure I.8: List of advantages and disadvantages of the drive mechanisms regarding the necessity for performing maintenance

I.2.4. Sustainability

Cylinders and wire ropes need hydro power units and electric motors to activate the driving mechanism. Also, inflatable systems need electric energy to activate the compressors. Hence, compared to buoyancy driven systems, these systems have a disadvantage with respect to sustainability. Wire ropes with drums have the additional need for lubrication for rotating components which can be exposed to the environment. A similar phenomena holds for cylinders which use hydraulic fluids that have a risk of leakage. Buoyancy and inflatable mechanisms have no disadvantages compared to the aforementioned ones, with respect to pollution risks. For this reason, buoyancy is graded perfectly, inflating mechanisms are graded good and cylinders and wire ropes are graded satisfactory.

In Figure I.9 the above mentioned advantages and disadvantages are summarised with the associated score points assigned. All concepts start with a perfect grade of ’5’ with respect to the sustainability and end up with a score in the MCA table by summing up the negative and positive score points to the starting value. The omission of pollution and external energy supply are not scored with additional score points, since the starting value of the grade is already ’5’.

	Advantages	Disadvantages
Cylinder		• Power units required that use external energy (-1) • Pollution risks for exposing lubricants to the environments (-1)
Wire rope		• Power units required that use external energy (-1) • Pollution risks for exposing lubricants to the environments (-1)
Buoyancy	• No power units required that use external energy (+0) • No pollution risks (+0)	
Inflation	• No pollution risks (+0)	• Power units required for compressors that use external energy (-1)

Figure I.9: List of advantages and disadvantages of the drive mechanisms regarding the sustainability in terms of pollution and energy consumption

I.2.5. Adaptibility

Adaptibility of the barrier should be considered as well when selecting the total working concept, including the gate type and drive mechanism. In the second evaluation in which is distinctively scored between the drive mechanisms per barrier type however, the adaptibility is not considered significantly distinctive between the drive mechanisms. Adapting the design would lead to more energy needed for the system to work properly, which is already covered in the criterion regarding sustainability. On top of that, in the design already is taken into account that the drive mechanism should be able to manage the extra weight for a potential adapted design, because the adaptation will only lead to a relatively small additional weight. The scores for adaptibility will be adopted from the first evaluation step regarding the multi-criteria analysis of the gate types, see Figure G.3.

I.2.6. Integrability

Integrability related to connection of the barrier with the surrounding area should be considered as well in the second evaluation for selecting the total working concept, which includes the gate type as well as the drive mechanism. However, this criterion does not affect the distinction of the evaluation of the drive mechanisms. The components responsible for the drive are recessed below the surface level. The integrability with respect to space usage on the contrary does affect the evaluation of the drive mechanism but that is already partly covered in the criterion regarding complexity. Thus, integrability in the second evaluation step only makes a distinction between the gate types, but should still not be omitted and thus be taken into consideration, because it is an important aspect when selecting the total concept for a self-closing flood barrier. The scores for integrability will be adopted from the first evaluation step regarding the multi-criteria analysis of the gate types, see Figure G.3.

J

Design of retaining height for barrier (overtopping/overflow)

J.1. Introduction

The self-closing flood barrier requires sufficient retaining height in order to keep the amount of wave overtopping and/or overflow within acceptable limits to prevent flooding with substantial consequences. The fault tree for this failure process is given in Figure J.1.

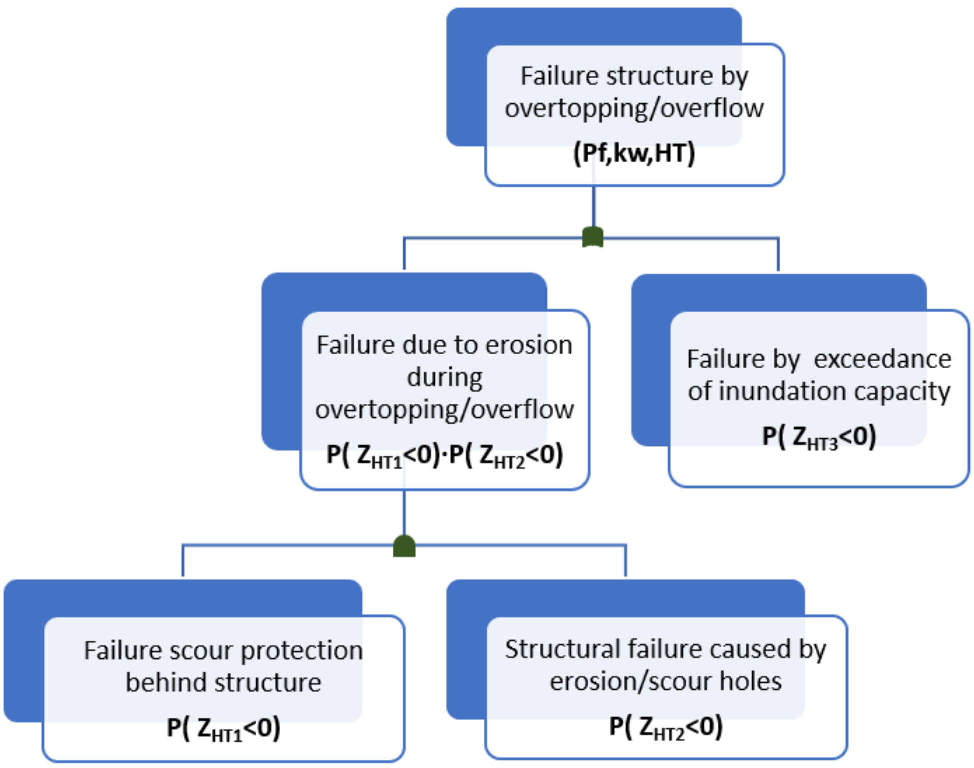


Figure J.1: Fault tree for failure process due to overtopping/overflow

The fault tree indicates that there is failure caused by insufficient retaining height if overtopping or overflow lead to an inundating amount of water that exceeds the storage capacity resulting in a flood event or if the soil surrounding the structure has been eroded by the flow due to wave overtopping or overflow causing the structure to fail due to instability.

In this verification, it is assumed that when erosion of the soil or bottom protection occurs, the structure instantly fails, meaning that $P(Z_{HT2} < 0) = 1.0$. The reason for this is that erosion is not preferable and moreover, this conservative approach leads to a safer design.

This means that the following holds:

$$P_{f,kw,HT} = P\ \{\ min\ (\ Z_{HT1}\ <\ 0\ ;\ Z_{HT3}\ <\ 0)\} \tag{J.1}$$

The limit state functions associated with these two failure events are:

$$Z_{HT1} = Q_c - Q_{os/ol} = q_c \cdot B_{sv} - q_{os/ol} \cdot B = u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv} - q_{os/ol} \cdot B \tag{J.2}$$

$$Z_{HT3} = V_c - V_{os/ol} = A_{kom} \cdot \Delta h_{kom} - q_{os/ol} \cdot B \cdot t_s \tag{J.3}$$

in which:

Z_{HT1}	[m³/s]	=	limit-state function erosion of soil or scour protection
Z_{HT3}	[m³]	=	limit-state function inundation capacity
Q_c	[m³/s]	=	maximum inundation rate (critical)
$Q_{os/ol}$	[m³/s]	=	discharge due to overtopping and/or overflow
q_c	[m²/s]	=	maximum inundation rate (critical) per unit width
$q_{os/ol}$	[m²/s]	=	discharge due to overtopping and/or overflow per unit width
B_{sv}	[m]	=	current width in gap
B	[m]	=	total width of self-closing flood barrier
u_c	[m/s]	=	critical flow velocity over soil or scrour protection
h_{bi}	[m]	=	inner water level behind the barrier (+ NAP)
h_{bb}	[m]	=	level of sill height (+ NAP)
V_c	[m³]	=	maximum water storage capacity
$V_{os/ol}$	[m³]	=	volume of water inundated by overtopping and/or overflow
A_{kom}	[m²]	=	water storage area
Δh_{kom}	[m]	=	maximum increase of inner water level behind barrier
t_s	[s]	=	the duration of the high water wave in the upper region of the Meuse

The load parameters, i.e. the overtopping and/or overflow discharge, are determined by Hydra-NL using the following underlying formula. The formula is applicable for a situation with only wave overtopping. Overflow will not be allowed in this case because that is not preferable in the relatively small residential area.

$$q_{os} = m_{os} \cdot \sqrt{g \cdot H_{m0}^3} \cdot e^{-3.0 \cdot \frac{h_{kh} - h}{H_{m0}} \cdot \frac{1}{\gamma_\beta \cdot \gamma_n}} \tag{J.4}$$

in which:
Hydra-NL calculates the hydraulic load level using a critical overtopping discharge and the failure probability requirement for the failure mechanism overtopping and/or overflow as input. The hydraulic load level corresponds to the minimum retaining height of the structure for which the probability of exceedance of the critical discharge is in compliance with the requirement.

$q_{os/ol}$ [m²/s] = discharge due to overtopping and/or overflow per unit width
m_{os} [-] = model factor for overtopping discharge = 0.13
H_{m0} [m] = significant wave height
h_{kh} [m] = retaining height of self-closing flood barrier (+ NAP)
h [m] = local water level in the Meuse (+ NAP)
γ_{β} [-] = influence factor for presence of dike structure reducing wave overtopping discharge
γ_{n} [-] = influence factor for oblique wave approach

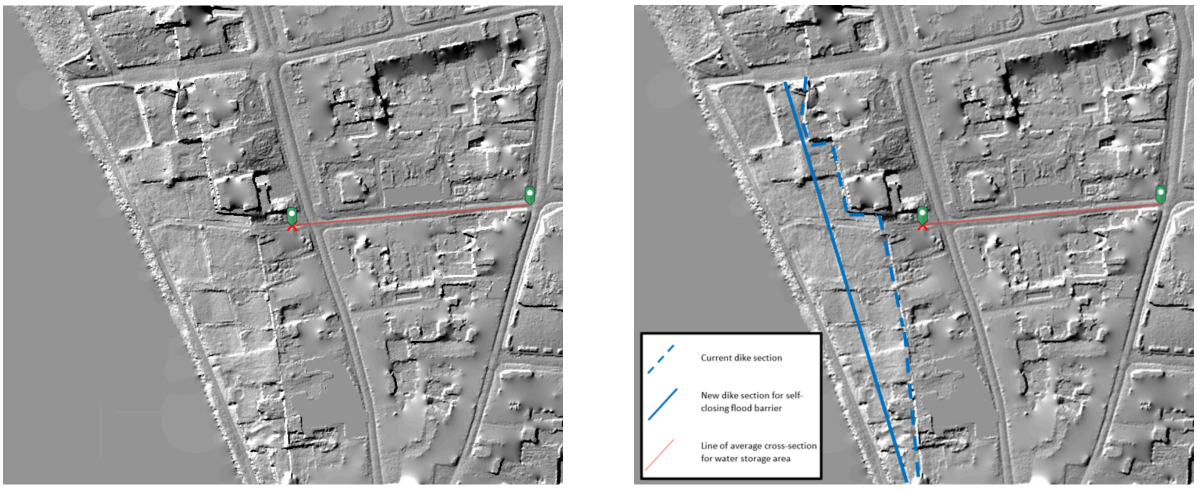
The critical overtopping discharge results from either the water storage capacity of the hinterland or the critical discharge where the scour protection or soil starts to erode. The governing failure mechanism, i.e. the strength parameters for each failure event that lead to the smallest critical discharge is the input for Hydra-NL. Each failure mechanism will be discussed below.

J.2. Water storage capacity

The water storage capacity follows from the area(s) in the hinterland A_{kom} in which water can freely flow to and will be stored in because of the relatively low surface elevation with respect to the rest of the area. Furthermore, the water storage capacity depends on the allowed inner water level increase Δh_{kom} before it is considered as a flood event with substantial economical damage and casualties.

For one postal code area with buildings and residents it is allowed to have an inner water level increase of $\Delta h_{kom} = 0.20$ m. (Rijkswaterstaat, 2023)

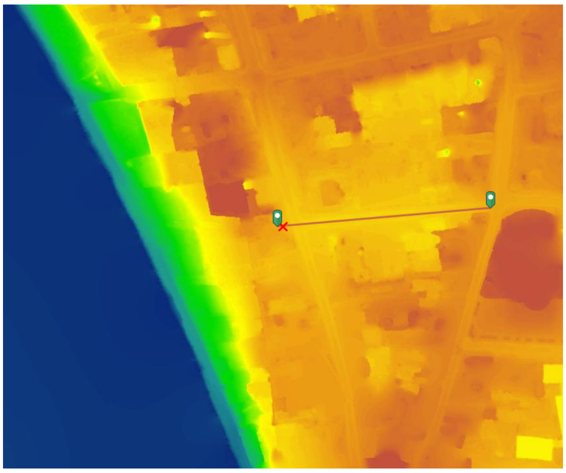
The water storage area is determined with the help of maps of the area and surface elevation data. In the Figures J.2, J.3 and J.4 these are shown.



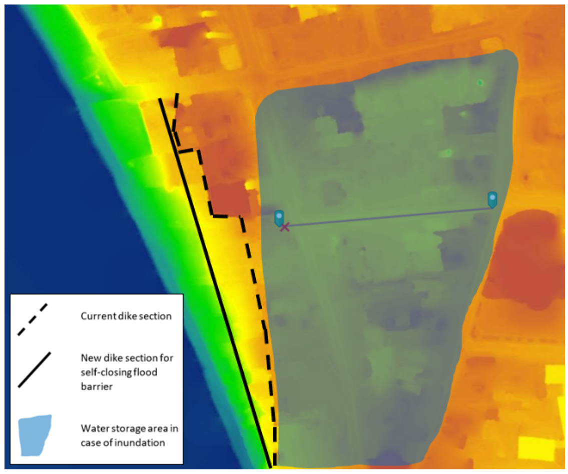
(a) DTM hillshade map of barrier location in Arcen and cross-section line

(b) DTM hillshade map of barrier location in Arcen with annotations

Figure J.2: DTM hillshade Maps of Arcen (AHN, 2023)



(a) DTM colour map of barrier location in Arcen and cross-section line



(b) DTM colour map of barrier location in Arcen with annotations

Figure J.3: DTM colour maps of Arcen (AHN, 2023)

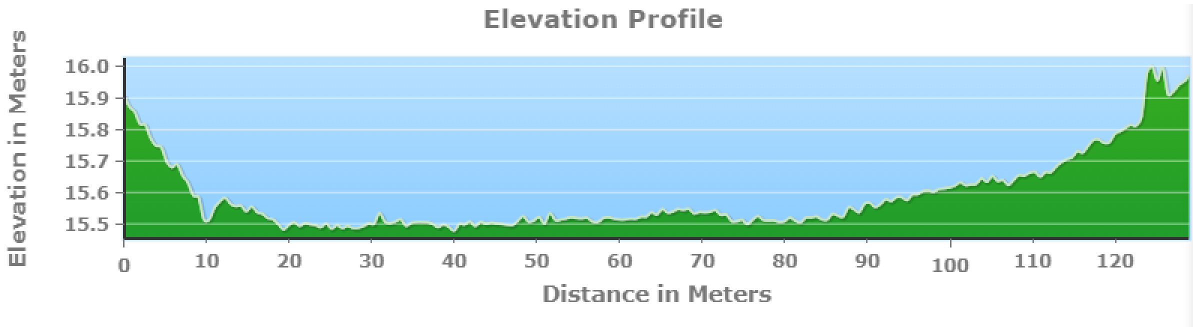


Figure J.4: Average cross-section of water storage area behind self-closing flood barrier in case of inundation (AHN, 2023)

The area (in blue) in Figure J.3 for the water storage is preliminary assumed to be 37000 m^3. An average cross-section of this area is depicted in Figure J.4, in which a basin is clearly visible. The span of the self-closing flood barrier for this dike section is $B = 230$ m. If it appears that the water storage capacity is the governing failure mechanism for determining the critical discharge, a follow-up step will be done in which the water storage area will be determined more accurately.

The critical discharge based on exceedance of the water storage capacity is equal to the inundation rate of the water storage area which depends in turn of the duration of the high water wave.

The typical or particular duration of a high water wave in the upper river region of the Meuse is obtained with the help of the software 'Water Level Gradient Tool'.

In Figure J.5 the graph is shown in which the duration can be determined.

Thus, the duration is approximately $t_s = 110$ hours, which is typical for a high water wave in a upper river region in the Netherlands.

From this follows that the critical discharge based on exceedance of water storage capacity is:

$$q_c = \frac{A_{kom} \cdot \Delta h_{kom}}{B \cdot t_s} = \frac{37000 \cdot 0.20}{230 \cdot 110 \cdot 3600} = 0.36 \frac{m^3}{s} / m \tag{J.5}$$

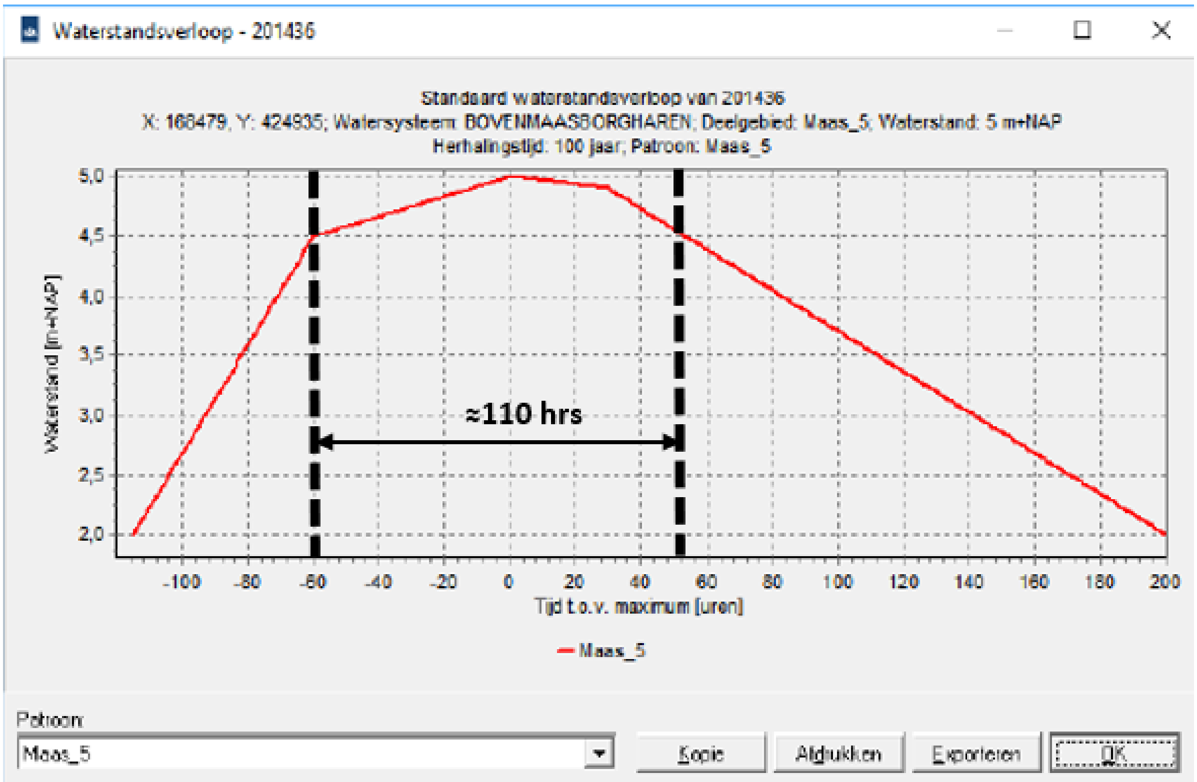


Figure J.5: Water level gradient during a particular high water wave in the upper region of the Meuse

J.3. Erosion of scour protection and/or soil

The critical discharge following from the strength of the scour protection or the soil depends on the load situation. Herein, a distinction is made between two load situations, namely directly loading due to a plunging jet or due to horizontal flow over the soil or the bottom protection. In this case, flow around the structure is not considered, because there is no inner water level which means there is no flow. Thus in this case, the load situation involving a plunging jet is only considered. However, a clear model for this phenomenon is not available and reference values for the critical discharge from the guideline are used. For cut-offs the guideline states that a critical value of $q_c = 0.05\frac{m^3}{s}/m$ may be used if the surrounding soil is paved. This is still not the case for the self-closing flood barrier, because it is located at private gardens which do not have bottom protection or pavements which are also not preferable to place in such locations. Thus, the critical discharge that should be used must be lower than $q_c = 0.05\frac{m^3}{s}/m$. Considering the maximum value for the design of grass dikes without scour protection where overtopping may occur, generally $q_c = 0.001\frac{m^3}{s}/m$ is considered as critical discharge for erosion. Although this is not exactly the same mechanism for the self-closing flood barrier, it still does give an indication to an assumed value for the discharge with a certain accuracy. The minimum insertable value for the critical discharge in Hydra-NL is also $q_c = 0.001\frac{m^3}{s}/m$ which is convenient. Important to note is, that this value could be higher in practice. Nevertheless, the effect of increasing the critical discharge to $q_c = 0.05\frac{m^3}{s}/m$ will be checked as well in order to see whether it is worth considering placement of scour protection anyways.

In either way, the critical discharge based on erosion is also the most governing one if compared to the critical discharge for the water storage capacity. The results of the sensitivity analysis are shown after the final calculations.

J.4. Calculation Hydra-NL

The calculation with Hydra-NL is done for several locations that are registered in the database of the dike section in Arcen. The eight locations for which the calculations of the hydraulic boundary conditions are performed, are marked with yellow in Figure J.6.

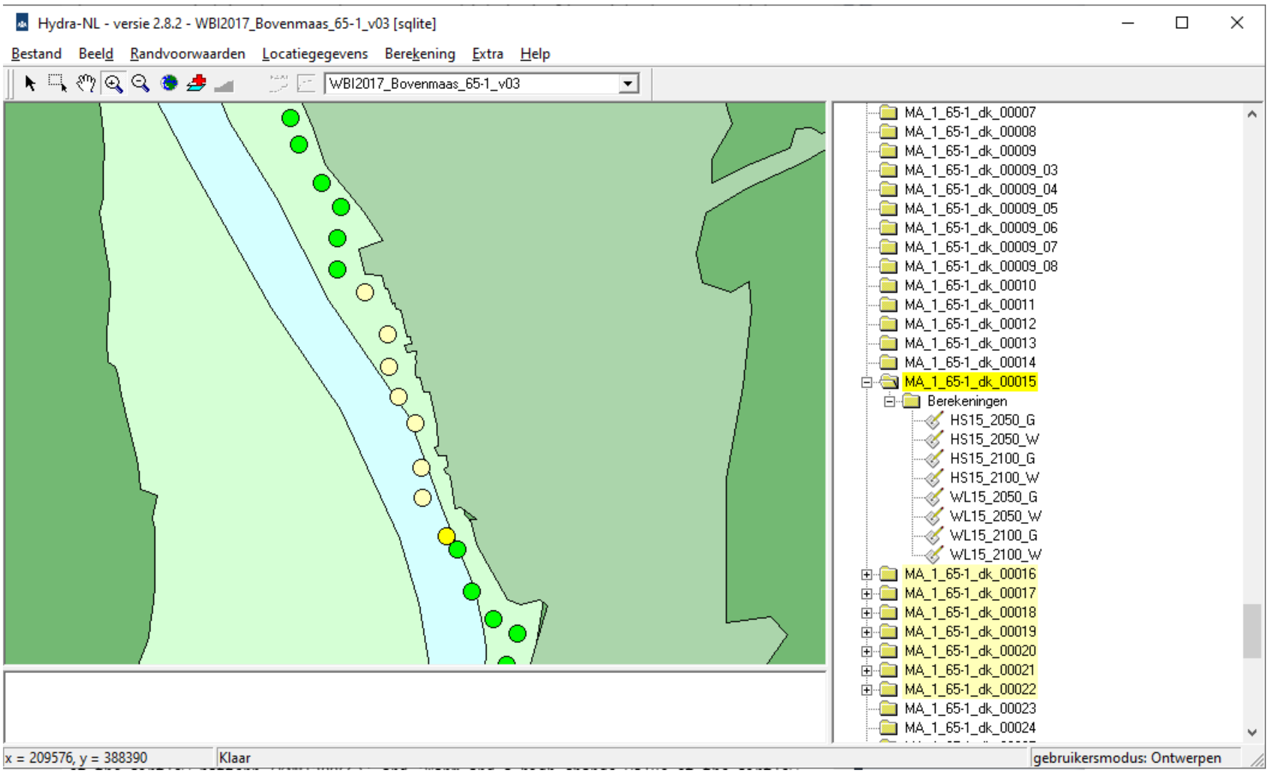


Figure J.6: Map in Hydra-NL with locations for which a dike segment calculation is available

The parameter input screen for the calculation in Hydra-NL is shown in Figure J.7. The software is Dutch so the important features used in the software with input are explained below. Each numbered item associates with the numbers in Figure J.7.

1. Type of calculation: several hydraulic boundary conditions can be calculated with Hydra-NL such as for example the expected water level in Dutch rivers along a several locations within a dike section. Besides this, also the significant wave height and hydraulic load level can be computed. The hydraulic load level is selected for the verification of the height of the barrier since it involves the expected water level and wave overtopping and/or overflow. Thus, the hydraulic load level refers to the minimum retaining height.

2. Critical overtopping discharge: as is described in the first feature, the hydraulic load level involves the contribution of wave overtopping for which a critical overtopping discharge should be entered. This follows from the functional requirements of the structure and its location. This value is determined to be $q_c = 0.001\frac{m^3}{s}/m$, but the results for $q_c = 0.05\frac{m^3}{s}/m$ will also be analysed.

3. Frequency: the frequency of occurrence of the hydraulic boundary condition to be calculated should be entered here. This follows from the failure probability requirement for the failure mechanism of overtopping/overflow for hydraulic structures within a Dutch dike segment which is derived from the Dutch standard. This requirement was determined in Chapter 3 in the Basis of Design, which was $P_{req,kw,HT} = 0.0024$ meaning that the frequency is 417 years.

4. Influence of wind at vertical wall: the influence of wind at a vertical wall can be taken into account in the calculation, which is ticked in this case.

5. Year to be considered for calculation: the functional lifetime of a structure determines for which year the boundary conditions should be computed. In Hydra-NL the years 2023, 2050 and 2100 can be selected. Other years require interpolation or extrapolation, which is necessary in this project, since the end of the functional lifetime is in 2065 and the end of the structural lifetime is in 2125.

6. Climate scenario: As a result of climate change, in addition to the average climate, the likelihood of extremes is changing. The KNMI regularly produces new climate scenarios for the Netherlands. They form the basis for research into the effects of climate change and adaptation to that change. These climate scenarios for a future climate must therefore provide information on both average change and change in extremes. KNMI distinguishes between four scenarios that differ based on global temperature change (moderate (G) and warm (W)) and airflow pattern change (low and high (+)). Hydra-NL uses only the two extreme scenarios for the prediction models for hydraulic boundary conditions: 'Moderately warm and a low change value of the airflow pattern (KNMI2006G)' and 'Warm and a high change value of the airflow pattern (KNMI2006W+)'. In this design study for a self-closing flood barrier, W+ will be the climate scenario for which the structure will be made following the 'Guideline Design Hydraulic Structures'.

7. Uncertainties: the calculation is able to be performed to take into account uncertainties in the model and the statistical data which is used. Both options are checked here.

As described earlier and in the basis of design, the climate scenario is W+, to determine the required hydraulic load level, i.e. retaining height for a self-closing flood barrier at the end phase of the lifetime of the concrete foundation. This is ultimately year 2125. This extrapolated calculation with results from Hydra-NL provides the inner dimensions for the vertical retaining height and thus the concrete foundation.

For the functional lifetime of each gate within the entire barrier, a period of 40 years is chosen, as was explained in the basis of design in Chapter 3. Thus, the year 2065 is the last year of the design lifetime after which adaptation to the design is applicable to comply to the requirements with the newly determined hydraulic boundary conditions for the next period. The hydraulic boundary conditions for 40 years with climate scenario W+ are the same for a lifetime of 100 years with climate scenario G. This means that if the climate change in the next 40 years does not follow the predictions of scenario W+, but the predictions of G, the design will be still acceptable regarding the requirements and the hydraulic boundary conditions. This would lead to no or less adaptability requirements, which is a preferred outcome.

In Figure J.8, a table is shown in which several hydraulic load levels (HLL) are calculated for years 2050 and 2100 for each climate scenario (marked in red). For the years 2065 and 2125 the values for the hydraulic load level are linearly interpolated and extrapolated based on the calculated values (marked in black).

In conclusion, the self-closing flood barrier with adaptability possibilities will be designed for a lifetime of 40 years (year 2065) with a hydraulic load level of 18.2 m + NAP , which results in a vertical height of the barrier of 2.83m relative to the terrain surface. With this height the design is verified to the failure mechanism of wave overtopping. The foundation is designed for the possibility of an adapted design with a maximum retaining height of 3.24m with an associated hydraulic load level of 18.6 m + NAP. The adaptability function of the self-closing flood barrier will be discussed in Section … . Associated with this hydraulic load level, is the water level in the Meuse, which is 17.84 m + NAP and the significant wave height, which is $H_{M0} = 0.26m$.

As was previously reported, the extent of the reduction of the hydraulic load level by increasing the allowed critical discharge from 1 l/s/m to 5 l/s/m is analysed as well. Increasing the critical discharge to 5 l/s/m results in a hydraulic load level reduction of approximately 0.16m. This is merely a reduction of the vertical height of the barrier of 6%.

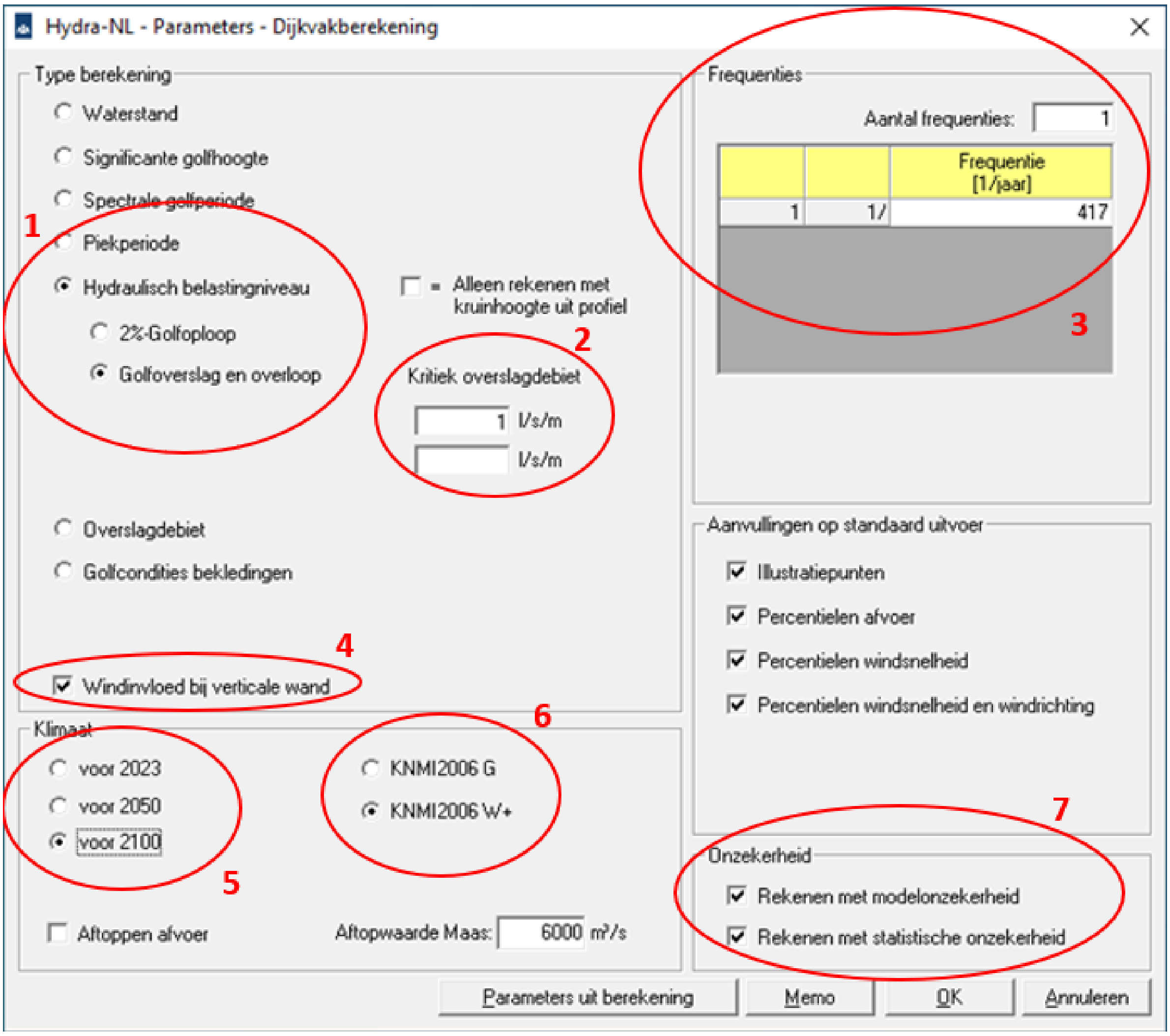


Figure J.7: Parameter input screen calculation Hydra-NL for hydraulic load level

		Climate Scenario KNMI2006					
		G			W+		
Year	Functional Lifetime	HLL (m+NAP)	Surface elevation (m+NAP)	Retaining height	HLL (m+NAP)	Surface elevation (m+NAP)	Retaining height
2050	25	18.0	15.4	2.56	18.1	15.4	2.73
2065	40	18.0	15.4	2.61	18.2	15.4	**2.83**
2100	75	18.1	15.4	2.73	18.5	15.4	3.07
2125	100	18.2	15.4	2.82	18.6	15.4	**3.24**

Figure J.8: Calculated hydraulic load levels per climate scenario G or W+ for the years 2050, 2065, 2100, 2125

In this design such an optimisation is not worth it to further investigate a better estimation of the critical discharge or to optimise the critical discharge by placing scour protection. The spatial quality is considered more important than economics in this case because of the location in private gardens as was described in the functional analysis in Chapter 2. It is not preferable to place scour protection in the gardens, taking up more space and depriving the liberty of the purpose of part of the property. For the construction also, this optimisation does not result in major changes. Thus, the critical discharge of 1 l/s/m with respect to erosion is considered still, leading to the results for the hydraulic load level in Figure J.8, which is also the most conservative approach and thus the most safest.

J.5. Verification exceedance water storage capacity

Hydra-NL also calculates the peak water level in the Meuse and the significant wave height at which the hydraulic load level is calculated. With these parameters, the retaining height, the wave overtopping formula and the water levels as a function over time, the amount of water inundated in the hinterland can be determined. This should be within the limits of the water storage capacity. This verification is done and the inundation is well within limits. This is not surprising, because the critical discharge in the calculations is 36 times smaller than the critical discharge determined for the water storage capacity. However, this verification is still performed, but it is not further elaborated on in this report, because of the extreme insignificance.

K

Design on probability of non-closure

Besides the global dimensions and the shape of the structure that are determined in the previous sections, it is also important for the functionality of the barrier to determine the maximum allowed failure probabilities of the components with respect to the closing process. The goal of this Appendix is to gain insight in the failure probabilities to which components should be designed for and thus to gain insight in the reliability of the structure with respect to the closure of the structure. The determination is based on a fault tree analysis.

K.1. Disclaimer

In this Appendix failure probabilities will be assigned for components related to the closure of the self-closing flood barrier to assign a certain reliability of the structure with respect to closure in order to perform this verification. It should be pointed out that these numbers are indicatively provided to gain insight in the order of magnitude of the maximum allowed failure probabilities and the critical parts in the design with respect to reliable closure. The numbers are more exactly specifiable when designing the closure components in detail. More importantly in this section is the set-up of the fault tree, the failure events and their relation with respect to each other.

K.2. Introduction

The self-closing flood barrier requires closing during an expected high water event. However, the closing process has a certain probability of failure, resulting in not closing, which leads in turn into a potential flood event if also the occurring water level exceeds a certain threshold. This failure probability depends on the selected gate type, the drive mechanism and the functional components of the latter two. Therefore in this section, the closing process of the self-closing flood barrier is designed in terms of failure probabilities to which must be complied to.

Firstly, the failure probability requirement for non-closure is stated and subdivided for one single closing gate. Then, the fault tree and the failure events of the closing process for one single gate are identified. This yields a failure probability requirement for the actual closing mechanism of the gate, but it depends on the closure demands per year, which must be determined first. Then by specifying the fault tree for the actual closing mechanism, insight is given in the failure probabilities to which the essential components and subprocesses should comply to, in such a way that the reliability of the closing process complies to the failure probability requirement regarding this failure mechanism for the closure demands per year.

K.2.1. Failure probability requirement from the standard for non-closure per dike section

The failure probability requirement for the failure mechanism 'non-closure' derived from the standard in the Dutch Water Act is:

$$P_{req,HS,NC} = \frac{P_{req,HS}}{N} = \frac{P_{max} \cdot \omega}{N} \tag{K.1}$$

where:

$P_{req,HS,NC}$	**=**	Failure probability requirement for non-closure for a hydraulic structure per year	$6.7 \cdot 10^{-5}$
$P_{req,HS}$	=	Failure probability requirement for non-closure for a dike section per year	$4.0 \cdot 10^{-4}$
P_{max}	=	Maximum permitted flood probability of the dike section (lower limit) per year	0.01
ω	=	Failure probability distribution factor for the failure mechanism in question	0.04
N	=	Length effect factor for the considered failure mechanism.	6.0

The length-effect factor herein is $N = 6.0$, because there are five dike sections within the dike segment with each a newly constructed self-closing flood barrier. Furthermore, in the current dike segment there is also already a pumping station present, which does not fall within the project area, but takes part in the entire dike segment and is therefore assumed to remain unaffected. This makes the total number of hydraulic structures within the dike segment for the length-effect factor to $N = 6.0$.

Dividing the barrier into individually closing structures within dike section

Within each dike section and thus the self-closing flood barrier of 240 m, the number of closing gate parts is set to $n_{gates} = 6.0$. This leads to a span per gate of approximately $40m$. This is a reasonable span length for gates with this hydraulic head, compared to reference projects (Daniel & Paulus, 2019). Decreasing the gate span length, i.e. increasing the number of gates leads to a more favourable condition for the structural design but more unfavourable for the reliability of closure, because there are more independently closing parts. Increasing the gate span length, i.e. decreasing the number of gates leads to a more unfavourable condition for the structural design but more favourable for the reliability of closure, because there are less independently closing parts. However, failure in such a case leads to larger consequences for the area. With $n_{gates} = 6.0$, i.e. a span length of 40 m both conditions seem to be balanced.

In conclusion the requirement to be verified for reliable closing of a self-closing flood barrier is $P_{req,HS} = 6.7 \cdot 10^{-5}$, which is to be divided into $n_{gates} = 6$ closing parts.

K.2.2. Fault tree for structure not closing

The fault tree for the structure not closing is given in Figure K.1 below. With the use of the fault tree the failure probability of the closing mechanism, $P_{f,CM}$, will systematically be derived.

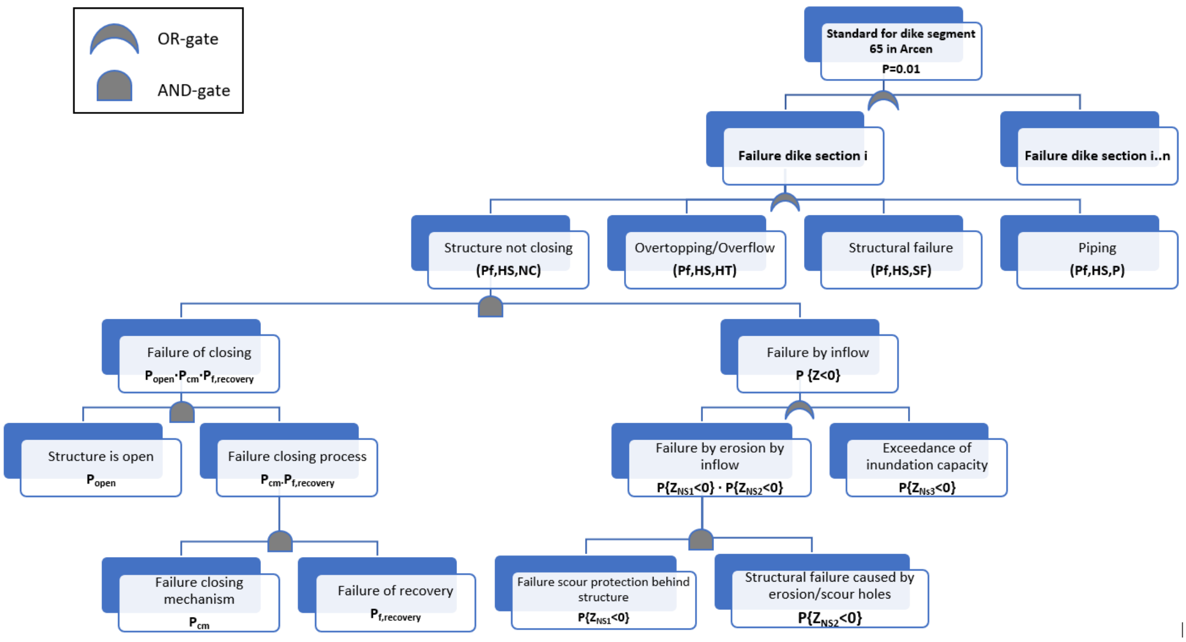


Figure K.1: Fault tree for the failure process related to reliable closure

There are five main events that determine the failure probability for not closing of the structure:

- The structure is open, with an expected high water event, which means that there is a closure demand. (P_{open})
- Failure of the closing mechanism which relates to the process from alarming to the technical closing $(P_{f,CM})$ and the failure of its corresponding recovery $(P_{f,recovery})$
- Failure of the scour protection behind the structure because of inundation $(P\ \{Z_{NC1} < 0\})$
- Structural failure caused by scour holes and erosion due to the scouring process $(P\ \{Z_{NC2} < 0\})$
- Exceedance of the inundation capacity in the area behind the structure $(P\ \{Z_{NC3} < 0\})$

Combining all events contributing to the failure probability of not closing of the self-closing flood barrier results in the following product of the contributing probability factors:

$$P_{f,HS,NC} = P_{f,CM} \cdot P_{open} \cdot P_{f,recovery} \cdot P\ \{Z < 0\} \tag{K.2}$$

, where the failure due to inflow, $(P\ \{Z < 0\})$, depends on either exceedance of inundation capacity (Z_{NC3}) or failure by erosion (Z_{NC1}). Herein, it is assumed that structural failure caused by erosion always occurs, thus $P\ \{Z_{NC2}\} = 1.0$.

In equation K.2, it is further assumed that the structure is always open $(P_{open} = 1.0)$ and recovery measures always fail $(P_{f,recovery} = 1.0)$, in order to take a conservative approach. The probability of failure of the closing mechanism $(P_{f,CM})$ and the probability of inflow $(P\ \{Z < 0\})$, which in fact is the number of closing demands per year, remain to be determined. All contributing factors in equation K.2 are elaborated below.

K.3. Probability of structure being open (P_{open})

The self-closing flood barrier should only be closed when a high water event approaches. This is the primary function of the structure. This means that the structure is always open and only requires closing when it should perform its water retaining function, unlike a sluice gate for example. This means that P_{open} = 1.

K.4. Probability of failure of recovery after a failed closure ($P_{f,recovery}$)

Recovery actions in this case would involve an emergency measure in which contractors and specialists are expected to be on site within two hours to recover the barrier from the failed drive, by for example lifting the gate with a winch truck, filling with the help of a water truck or removing obstacles. This would be assuranced by a signed contract between the parties. Including the failure probability of a recovery of a failed closure will have a positive contribution on the failure probability of not closing. In this design verification this will not be taken into account, even though recovery actions are often part of a closure protocol for hydraulic structures. The reason for this is, because it is preferable to tighten the failure probability requirement of the closing mechanism to increase the reliability and create sufficient margin for additional safety. So this implies that the failure probability for a recovery of a failed closure is, $P_{f,recovery}$ = 1, implying that the recovery always fails, which means that the closing mechanism relies fully on the closing mechanism itself consisting of the primary and back-up drive. This is a conservative approach, since in the case of a signed contract and the fact that yearly in practice many lifting operations take place. So a failure probability of 1 is therefore not considered realistic in practice.

K.5. Determination of the required probability of inflow ($P\ \{Z < 0\}$; closure demands per year)

When the self-closing flood barrier is not fully closed in case a high water event occurs, the area of the hinterland will be inundated by a flow through the opening(s) at the locations where the individually closing part(s) of the structure failed to close. The rate at which the hinterland floods depends on the size of the storage capacity and the discharge through the opening(s) which depend in turn on the size of the opening and the outer water level.

The probability of failure of not closing depends on the number of closure demands per year, because the structure should only close in case of a high water event, which has a certain probability of occurrence of its own. Failure of closing the water barrier in combination with a water level in the Meuse that does not have a consequence of flooding the hinterland or erosion of the soil near the structure is not labelled as failure of closing.

Thus, this contribution to the failure probability relies on the probability of occurrence of a certain outer water level, where the area of the hinterland will be inundated by a flow through the opening(s) at the locations where the individually closing part(s) of the structure failed to close. More specifically, it depends on the probability of a certain outer water level which leads to exceedance of the water storage capacity in the hinterland by an inflow through the opening(s) or which leads to an inflow with a flow velocity exceeding the critical flow velocity of the soil or bottom protection causing erosion of the soil near the structure. The governing situation, i.e. the situation with the highest probability of occurrence, will be considered.

From the fault tree in Figure K.1, it can be seen that probability of erosion only is taken into account with the conditional probability that the structure fails given that erosion occurs. With other words it is only considered as failure if the structure fails by for example instability after erosion of the soil near the structure. For conservative reasons it is assumed that whenever erosion of the soil occurs always structural failure occurs as well. So the conditional probability of structural failure given erosion is assumed to be $P\ \{Z_{NC2} < 0\} = 1$.

For the determination of which of the two events is governing, insight in the other limit state functions

(Z_{NC1} and Z_{NC3}) is necessary. These were already presented in the verification of the height of the barrier in Appendix J:

$$Z_{NC1} = Q_c - Q_{ot/of} = q_c \cdot B_{sv} - q_{ot/of} \cdot B = u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv} - q_{ot/of} \cdot B$$

$$Z_{NC3} = V_c - V_{ot/of} = A_{kom} \cdot \Delta h_{kom} - q_{ot/of} \cdot B \cdot t_s$$

in which:

Z_{NC1}	[m³/s]	=	limit-state function erosion of soil or scour protection
Z_{NC3}	[m³]	=	limit-state function inundation capacity
Q_c	[m³/s]	=	maximum inundation rate (critical)
$Q_{ot/of}$	[m³/s]	=	discharge due to overtopping and/or overflow
q_c	[m²/s]	=	maximum inundation rate (critical) per unit width
$q_{ot/of}$	[m²/s]	=	discharge due to overtopping and/or overflow per unit width
B_{sv}	[m]	=	current width in gap
B	[m]	=	total width of self-closing flood barrier
u_c	[m/s]	=	critical flow velocity over soil or scrour protection
h_{bi}	[m]	=	inner water level behind the barrier (+ NAP)
h_{bb}	[m]	=	level of sill height (+ NAP)
V_c	[m³]	=	maximum water storage capacity
$V_{ot/of}$	[m³]	=	volume of water inundated by overtopping and/or overflow
A_{kom}	[m²]	=	water storage area
Δh_{kom}	[m]	=	maximum increase of inner water level behind barrier
t_s	[s]	=	the duration of the high water wave in the upper region of the Meuse

Firstly considering the limit state function for the bottom protection, the critical flow velocity of the soil or scour protection is one of the strength parameters that determines the strength. The area has no bottom protection and consists of fine sand which has a critical flow velocity of $u_c = 0.1 m/s$ (Rijkswaterstaat WVL, 2021). This is extremely low, i.e. highly erodible. With this maximum flow velocity, an outer water level equal to the ground surface level at the location of the barrier would already be too much and would lead to erosion. With this knowledge, the second limit state function does not need to be considered, because any type of flow through an opening of a failed to close gate part is not allowed, which means that no inundation is allowed as well. To conclude, erosion of the soil (Z_{NC1}) is the governing situation for the determination of the closure demands and the allowed water level is equal to the ground surface level at the location of the barrier, which is NAP + 15.4 m. The probability of occurrence of this water level leads to the right number of closure demands per year.

For this, the frequency line of the water level in the Meuse at the location of the structure is required. This is obtained from Hydra-NL. Hydra-NL only shows occurrences with a return period of 10 years and more. A return period of 10 years ($Log(T) = 1$) is too large for a water level of NAP + 15.4 m. The frequency line is extrapolated with the use of a fitted line.

The function that describes this line is given in Figure K.2 and follows the frequency line very well. On the vertical axis the water level is given in meters relative to NAP and on the horizontal axis the log function of the return period in years is given. The solid blue line gives the frequency line as exported from Hydra-NL. Note that the line starts at $Log(T) = 1$, which corresponds to a return period of 10 years. The dotted blue line is the extrapolated line generated with the help of Excel.

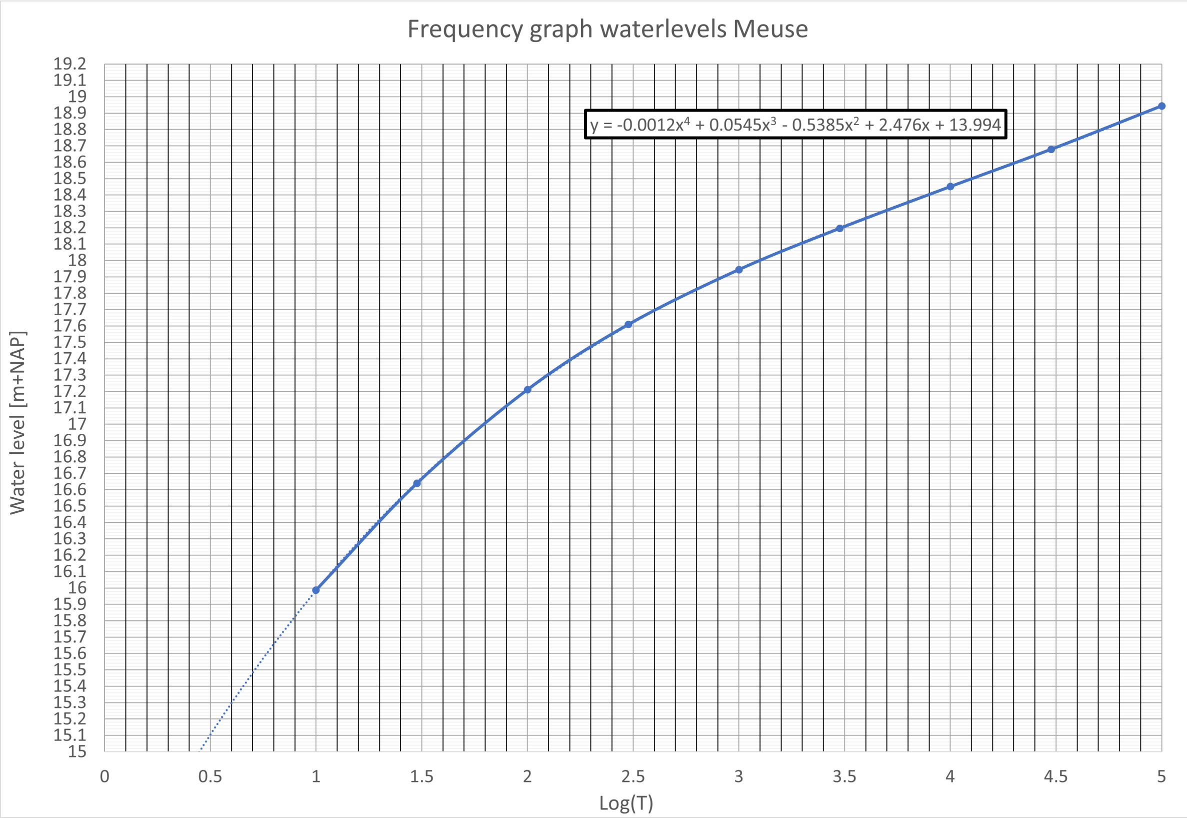


Figure K.2: Extrapolated frequency line of water levels (in m with respect to NAP) in the Meuse with Log(T) on the horizontal axis and the water level on the vertical axis

A water level of NAP + 15.4 m occurs at $Log(T) = 0.65$, which results in a return period of T = 4.47 years on the average and thus a probability of $P\ \{Z < 0\}\ ) = 0.22$.

Important to note here is that the probability of occurrence is determined with current statistics and prediction models, which can have a different outcome in five years. For example, due to climate change, future water levels in the Meuse could be higher, leading to a higher number of closure demands. This would mean a higher probability of non-closure per year, which in turn would result in stricter requirements for the design of the closing mechanism.

Now that the closure demands per year is determined, the last contributor in Equation K.2 can be determined, which is the failure probability of the closing mechanism $P_{f,CM}$. This is elaborated in the following paragraph.

K.6. Determination of the required failure probability of the closing mechanism

This section shows the determination of the required failure probability of the closing mechanism resulting from the other contributing factors taken into account as determined previously to the failure probability requirement for the failure mechanism of not closing. The requirement derived from the standard of the Dutch Water Act as determined with length-effect factor of $N = 6$ is:

$$P_{req.HS,NC} = 6.7 \cdot 10^{-5}$$

The probability of occurrence of not closing is determined with Equation 4.3, from which only $P_{f,CM}$ is the unknown:

$$P_{f,HS,NC} = P_{f,CM} \cdot P_{open} \cdot P_{f,recovery} \cdot P\ \{Z < 0\} = P_{f,CM} \cdot 1.0 \cdot 1.0 \cdot 0.22 = 6.7 \cdot 10^{-5}$$

which means that the verification reliable closing is in agreement, when the last contributing factor, the required probability of failure of the closing mechanism is:

$$P_{f,CM} = \frac{P_{f,HS,NC}}{P_{open} \cdot P_{f,recovery} \cdot P\ \{Z < 0\}} = \frac{6.7 \cdot 10^{-5}}{1.0 \cdot 1.0 \cdot 0.22} = 3.03 \cdot 10^{-4} \tag{K.3}$$

K.7. Design on required probability of failure of closing mechanism (actual closing process) ($P_{f,CM}$)

In this section the design of the self-closing flood barrier is assigned with failure probabilities with respect to the closing mechanism, consisting of gate closure, the drive mechanism and failure events for these processes. From the analysis eventually follows what is required for the design to meet the failure probability requirement for the closing mechanism. The failure probability requirement for the closing mechanism as determined in the previous section is $P_{f,CM} = 3.03 \cdot 10^{-4}$. The barrier consists of six individually closing gates, $n_{gates} = 6$, which means that per gate the failure probability requirement for the closing mechanism results in $P_{f,CM,i} = 5.05 \cdot 10^{-5}$.

The failure probability of the closing mechanism of a generic hydraulic structure takes into account four subprocesses concerning:

1. Alarming
2. Mobilisation
3. Operating
4. Technical failure

However, for a self-closing flood barrier the first three subprocesses are irrelevant because these involve human activities which are not the case with the self-closing flood barrier. Thus, the emphasis for the failure probability of the closing mechanism is on the technical failure only.

Per individual closing gate part the probability of failure of the closing mechanism can be translated to a fault tree showing the contributing failure processes. There are two methods for achieving this, namely a standardised method with a generic fault tree and an advanced method with a customised fault tree specifically for this design of a self-closing flood barrier. Each method is elaborated for comparison and briefly described below with the outcomes regarding the design with respect to the closing mechanism.

Important to note up front is that the standardised method results in an overall failure probability for non-closure which is not acceptable, i.e. leading to non-compliance with the requirement. For this reason, the advanced fault tree analysis is necessary to perform to customise the design and the associated closing protocols to satisfy the failure probability requirement of the closing mechanism of the barrier. The advanced method, which is leading in this thesis is described firstly, after which the standardised method follows.

K.7.1. Advanced fault tree analysis

The customised fault tree is depicted in Figure K.3.

The fault tree has the same format as the standardised fault tree in order to compare both with respect to each other. The fault tree is only elaborated for one single closing gate part of the entire barrier. For each single closing gate part, the fault tree is the same because all gate parts are considered identical. The total failure probability for non-closure of the self-closing barrier is the sum of each failure probability per gate part. Below, each branch representing a contributing failure process is discussed further for closer understanding of the assigned probabilities.

Gate failure: during closure movement

During a closure demand, failure of closing can also occur due to, for example, tilting of the gate which is made possible by, for example, littering and siltation in the concrete floatation chamber. The probability assigned to this is $1.0 \cdot 10^{-5}$ which is supported by an ANSI/ANS-58.21-2007 norm method (ANSI = American National Standard Institute) used by Rijkswaterstaat (van Bree & Casteleijn, 2017). Another possibility for failure during closure, as can be seen in the fault tree, could be that an obstacle, such as a parked car, is present on the gate, impeding the closure. In this case it would have the primary consequence that the gate would be blocked from its closing movement, rather than misalignment of the gate. The likelihood for this case would be $1.0 \cdot 10^{-4}$ resulting from the same ANSI analysis. However, the probability assigned to this is $1.0 \cdot 10^{-5}$, which is a reduction by a factor 10. The reason for this is because an obstacle being a parked car is less likely in the area, but particularly because during an anticipated high water event, as part of an inspection regime applicable to the control of this barrier, an inspection is performed two hours prior to a closure demand to remove obstacles such as large flower pots or other physical obstructions, if any. The probability of occurrence of $1.0 \cdot 10^{-5}$ here is therefore the probability of the physical obstruction combined with a situation when no intervention is possible leading to failure of closure.

Gate failure: before closure movement

Noticeable failure

Noticeable failure means that an event occurs that leads to disuse of the retaining structure, which can be noticed almost immediately in such a way that it can be notified before a high water event. For noticeable failure events, it is assumed that an obstacle is present on top of the retaining structure which may cause deformations or misalignment such that the closure mechanism does not proceed as intended resulting in an impeded closure. Based on an ANSI/ANS-58.21-2007 norm method, which is also recommended by the guideline for risk controlled maintenance and control by Rijkswaterstaat, the likelihood of occurrences of external events can be quantified. As mentioned for the likelihood of a parked car on a non-closed barrier according to this ANSI analysis, a probability per closure demand of $1.0 \cdot 10^{-4}$ is plausible. Considering this event and its likelihood and knowing that the location of the self-closing barrier in Arcen is not accessible to cars and thus also parked cars located at the barrier, the probability of this event occurring would be lower than $1.0 \cdot 10^{-4}$. In fact, the area concerns private gardens, so obstacles are mainly possible in the form of utensils, garden accessories or other objects that can be used in private outdoor areas. For these potential obstacles, the load is much lower and so would be the probability of noticeable failure for non-closure. Therefore, a failure probability per closure demand of $1.0 \cdot 10^{-5}$ is considered, which is ten times smaller than the likelihood for a parked car on a barrier opening according to the ANSI analysis.

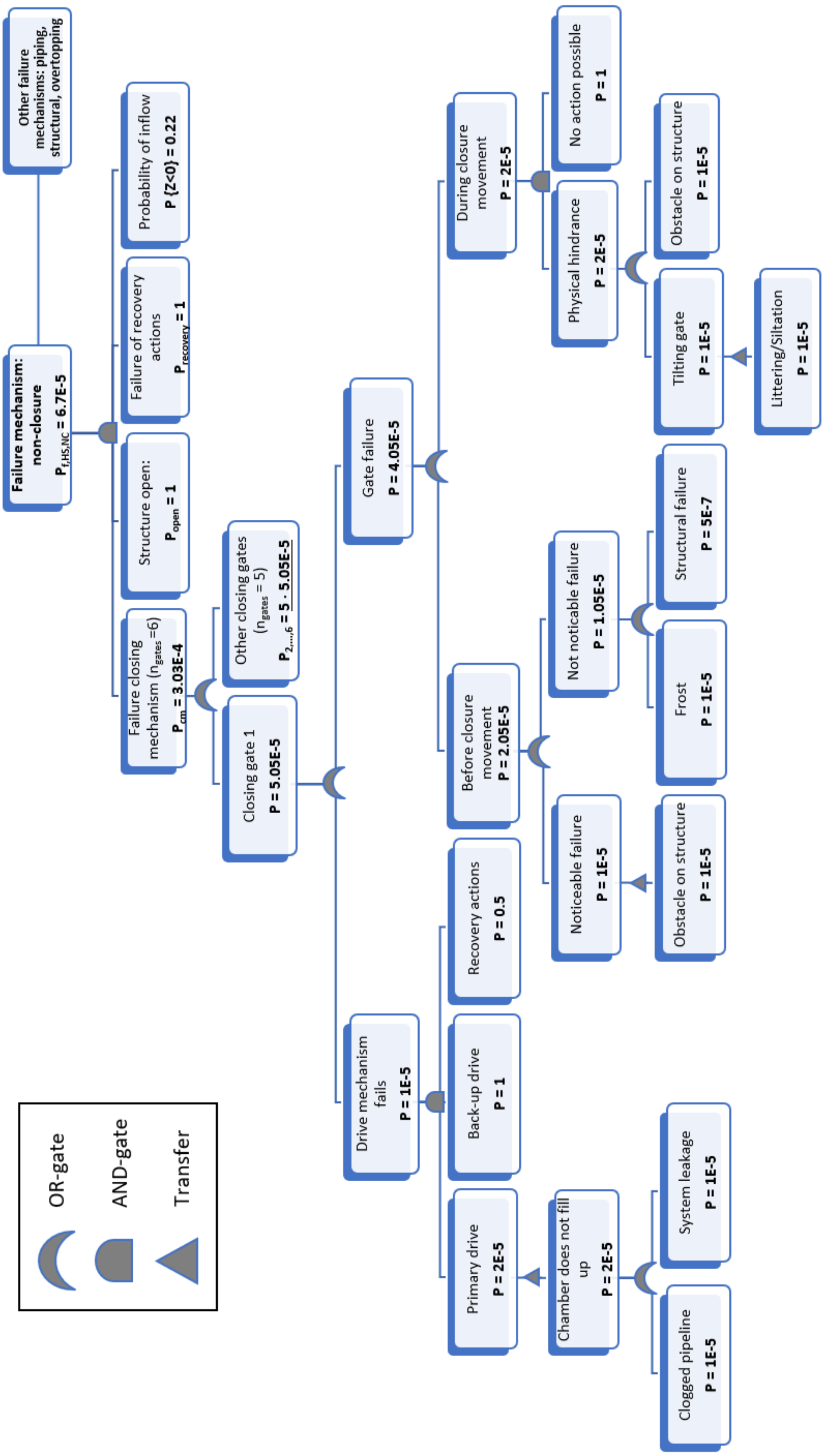


Figure K.3: Fault tree for the failure of the closing mechanism

Not noticeable failure
Non-noticeable failure involves the events which specifically cannot be noticed before a high water event, in contrast to noticeable failure events.

1. Frost
An event in this category would be for example extreme frost. The retaining structure, guidance rollers and seals between the gates may experience extreme frost. This can lead to jamming of the structure or impeded closure due to ice formation which is not immediately noticeable. According to the ANSI analysis, a failure probability per closure demand of $1.0 \cdot 10^{-4}$ is plausible for this event. However, the risk of frost-related components of the self-closing barrier can be executed with a frost-free coating, reducing the risk of frost. The likelihood of such an event is thus lower than $1.0 \cdot 10^{-4}$. In this fault tree it reduced by a factor of 10 to $1.0 \cdot 10^{-5}$.

2. Structural failure
Another not noticeable event would be corrosion or a biological attack to the steel, which would be categorised as structural failure. This failure process belonging to non-closure refers to special or unforeseen load cases that can lead to structural failure even before a closure demand occurs, i.e. in an open state. The structural design on strength and stability is elaborated for situations in a closed position which is not evaluated here. Structural failure in an open position however can lead to failure of closing. The probability of this is derived from other reference projects such as the sluice complex at Meppelerdiep, where the probability of suchlike structural failure is set to a value with order of magnitude of $1.0 \cdot 10^{-7}$.

Drive mechanism failure: primary
The primary drive for each individually closing part of the total gate involves a pipeline connected to the Meuse River that, at a signaling water level, allows water to enter the floatation chamber. Failure of the primary drive implies not filling of this floatation chamber, hence non-floating of the barrier and thus failure of closure. This can happen due to two events, namely, a clogged pipeline or a leak in the pipeline or anywhere in the system due to damage. From the failure database of Rijkswaterstaat, a clogged valve statistically has a failure probability per closure demand of $1.0 \cdot 10^{-5}$. This is considered reasonable for a clogged pipeline as well, but the design of pipes can include metal grids and filter to prevent debris from flowing in, justifying this assigned failure probability. Further, for the second event, where there would be a leak in the system, the average probability is taken from the likelihood of a leak of several components which are included in the failure database of Rijkswaterstaat. The probability of occurrence of a leak in the pipeline is assigned to be $1.0 \cdot 10^{-5}$. This probability includes also the applicability of an inspection regime, where the pipelines are checked twice a year for a leak and right before a closure demand, which can justify the feasibility of a design with a failure probability for leakage of $1.0 \cdot 10^{-5}$.

Drive mechanism failure: back-up drive
The backup drive involves filling the system with a secondary pipe. For conservative considerations, a failure probability of 1 is assigned here.

Drive mechanism failure: recovery
The probability of the failure of recovery actions is assumed to be 0.5. Recovery in this case involves unclogging a pipe or installing a pump from a water truck to the pipe to compensate the leakage of the system. The probability of these events failing is assigned to be 0.5. This means that in 50% of the cases when a recovery action is required, failure still occurs to lift the barrier into retaining position. This is a rather conservative approach to ensure a certain reliability of the primary drive. However, a 100% probability of failure of the recovery measure is not assigned, seeing that it is not realistic in practice and therefore not considered reasonable in this fault tree analysis. An additional advantage of this conservative approach is that in a detailed follow-up analysis the primary drive still has a certain margin in the failure probability, since in reality the failure probability of recovery is likely to be smaller.

Underlying equations for total failure probability of closing mechanism
The failure probability for the closing mechanism of the entire barrier is:

$$P_{f,CM} = \sum_{i=1}^{i=6} P_{f,CM,i} \qquad ; i = \{1, .., 6\} \tag{K.4}$$

Since it is considered that each of the six independently closing gates within the barrier are identical, the equation can be simplified to:

$$P_{f,CM} = n_{gates} \cdot P_{f,CM,i} \qquad ; n_{gates} = 6 \tag{K.5}$$

The failure probability for the closing mechanism of one closing gate as part of a total of six gates within the entire barrier is:

$$P_{f,CM,i} = P_{f,drive} + P_{f,gate} \qquad ; i = \{1, .., 6\} \tag{K.6}$$

in which:

$$P_{f,drive} = P_{f,primary} \cdot P_{f,back-up} \cdot P_{f,recovery} \tag{K.7}$$

$$P_{f,gate} = P_{f,during\ closure} + P_{f,before\ closure} \tag{K.8}$$

which becomes:

$$P_{f,CM,i} = P_{f,primary} \cdot P_{f,back-up} \cdot P_{f,recovery} + P_{f,during\ closure} + P_{f,before\ closure} \tag{K.9}$$

$$P_{f,CM,i} = 2.0 \cdot 10^{-5} \cdot 1.0 \cdot 0.5 + 2.0 \cdot 10^{-5} + 2.01 \cdot 10^{-5} = 5.01 \cdot 10^{-5}$$

As a check if the design with assigned failure probabilities comply to the requirement:

$$P_{f,CM} = n_{gates} \cdot P_{f,NS,i} = 6.0 \cdot 5.01 \cdot 10^{-5} = 3.01 \cdot 10^{-4} \tag{K.10}$$

$$P_{f,CM} = 3.01 \cdot 10^{-4} < 3.03 \cdot 10^{-4} \tag{K.11}$$

As can be noticed, there is a difference of $2 \cdot 10^{-6}$ with the failure probability requirement of the closing mechanism, which can be added to one or more of the assigned failure probabilities of the components. However, this is in this report not of significance and will be neglected further.

K.7.2. Standardised fault tree analysis with score tables

In this analysis, the standard method is solely used to validate the outcome of the advanced fault tree analysis. The standardised fault tree is pictured in Figure K.4. This is a generic method of determining the failure probability due to non-closure for any closable hydraulic structure in the Netherlands. The standard method has a lower limit of the failure probability, meaning that specific measures to increase the reliability of the structure, for example optimising on critical components or tightening the inspection and maintenance regimen is not taken into account within the standard method (Casteleijn & van Bree, 2017). The standard method is helpful as a tool to simply and quickly determine the failure probability regarding the closing process if the design closely resembles a standard hydraulic closing structure and if there is enough tolerance in the failure probability requirement for closing that a detailed analysis might not be required (Casteleijn & van Bree, 2017).

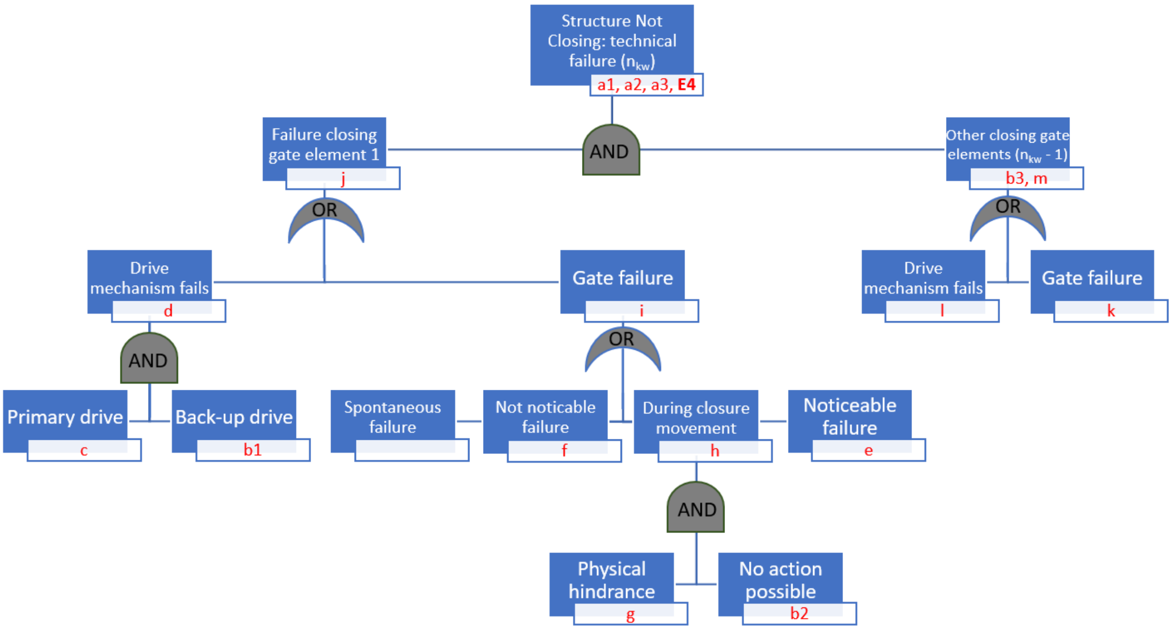


Figure K.4: Standardised fault tree for failure process related to reliable closure

As can be seen, a distinction is made with the several branches as mentioned before in the advanced fault tree analysis. Also, note that this fault tree only considers the technical failure and not alarming, mobilisation and operation as is normally the case with this method for a closing water retaining structure. The maximum assignable failure probabilities regarding these processes involving human activities are relatively low anyways. The determination of the failure probability based on this fault tree is achieved by filling in a score table with questions which results in a final score that is related to a failure probability.

Below the score table is shown which is filled in to achieve a final score leading to failure probability for not closing.

Part	Question		Answer	Score	Score	Comment
A	a1	Is there a maintenance plan available for the water retaining structure and is it being adhered to?	yes no	0.5 0	0.5	Once a year maintenance to the barrier
A	a2	Is the primary and if applicable the secondary retaining structure being checked at least two times a year and the closing mechanism at least once a year being tested, including all corresponding drive mechanisms?	yes no	1.5 0	1.5	-
A	a3	Are the findings of the checks, tests and actual closures being reflected and improvements being redirected in the mobilisation regulation and operation protocol or if necessary to the closing structure itself?	yes no	0.5 0	0.5	-
Drive	c	Is the closing structure manually closeable?	yes no	0.5 0	0.5	Manually hoistable
Drive	b1	Is a second drive mechanism available?	yes no	1 0	1	Chamber fillable with help of water pump
Drive	d	Drive mechanism fails (intermediate score)	c+b1	min 0 max 1.5	**1.5**	-
Retaining wall	e	Is there any significant risk of noticeable failure of the retaining structure?	yes no	1 1.5	1	Obstacle on barrier
Retaining wall	f	Is there any significant risk of not noticeable failure of the retaining structure?	yes no	1.25 1.5	1.25	Frost, Corrosion, Biological attack
Retaining wall	g	Is there any significant risk of obstruction or impediment causing the closure to fail?	yes no	0.5 1	0.5	Littering, Siltation
Retaining wall	b2	Is there anticipated in the closure protocol on this risk of obstruction or impediment?	yes / N/A no	0.5 0	0.5	Inspection before expected high water, possible restoration/repair
Retaining wall	h	Failure during closure: impediment (intermediate score)	g+b2	max 1.5	**1.0**	-
Retaining wall	i	Retaining structure 1 fails (intermediate score)	min(e,f,h)	min 0.5 max 1.5	**1.0**	-
1st closure	j	Closure retaining structure 1 fails	min(d,i)	min 0 max 1.5	**1.0**	-
2nd retaining wall	b3	Is there a second independent retaining structure being operational if the primary retaining structure is not able to close? If yes: answer question k and l for the second retaining structure.	yes no	0.75 0	0	-
Retaining wall	k	Is there any significant risk of failure of the retaining structure?	yes / N/A no	0 0.25	0	-
Drive	l	Is the second closing structure manually closeable?	yes no / N/A	0.25 0	0	-
2nd closure	m	Drive mechanism 2 fails (intermediate score)	min(b3+k,b3+l)	min 0 max 1	**0**	-
	E4	Water retaining structure not closing by technical failure and failure of recovery actions	a1+a2+a3+j+m	max 5	**3.50**	-

Figure K.5: Score table related to standardised fault tree to determine

As a result, the final score E4 for the self-closing flood barrier not closing by technical failure and failure of recovery actions is E4 = 3.50. The corresponding failure probability then is determined by:

$$P_{f,NS,technical\ failure} = 10^{(-E4)} = 10^{-3.50} = 3.2 \cdot 10^{-4} \quad \text{(K.12)}$$

This is only for one individual closing gate part. The entire barrier consists of six ($n_{gates} = 6$) individual closing gate parts resulting in a total failure probability for not closing:

$$P_{f.NS} = 1.9 \cdot 10^{-3}$$

K.7.3. Conclusion of the difference between custom fault tree analysis and standard fault tree with score table

According to the standard fault tree with score tables, the failure probability is $P_{f.NS} = 1.9 \cdot 10^{-3}$. This is about six times higher than the required failure probability for the closing mechanism. This is nonetheless expected and acceptable because the standard fault tree with score tables is more conservative because it is a more generic method of determining the failure probability due to non-closure for any closable structure. Moreover, the standard method does not allow the failure probability for the closing mechanism to be lower than the minimum of 10^{-5} for one individually closing part. For example, if the structure were to be designed with a perfectly reliable closing mechanism, the standard method would still have the same probability of failure which would be inconsistent with the reality. Optimising on critical components or on the inspection and maintenance regimen, which would lead to a lower probability of failure, is not possible with the standard method. However, with a specific fault tree analysis this is actually possible, which allows the engineer to have more liberty in the design choices and thus similarly in the associated failure probability. The standard method is helpful as a tool to simply and quickly determine the failure probability regarding the closing mechanism if the design closely resembles a standard hydraulic closing structure and if there is enough tolerance in the failure probability requirement for closing that a detailed analysis might not be required.

Looking further at other difference analyses of reference projects for the failure probability due to non-closure, it can be seen that a difference with a factor of 6 is actually quite reasonable and resembles these other difference analyses that resulted in a factor between 1 and 10 between the advanced and standard fault tree analyses (van Bree & Casteleijn, 2017.

In conclusion, comparing the two methods, it is clear that with the advanced fault tree analysis designing on the failure probabilities is achievable to have an acceptable probability of failure regarding the closing mechanism.

Conclusion on design with failure probability of closing mechanism

As was mentioned in the disclaimer in the beginning of this section, the assigned failure probabilities provide an order of magnitude which are considered reliable based on a comparison with failure probabilities of external events from the ANSI/ANS 58.21-2007 norm method (ANSI = American National Standard Institute) used by Rijkswaterstaat (van Bree & Casteleijn, 2017). This means that a design for a self-closing flood barrier regarding the reliability of the closure mechanism as such is considered feasible. Additionally, in the fault tree of Figure K.3 can be seen that the failure probability of the back-up drive and the recovery action are assumed to be very high, which in reality would not be the case. This is merely done to take a conservative approach and to show that the failure probability of the primary drive mechanism has a certain margin in the design.

Furthermore, from the resulting fault tree in Figure K.3, it can be concluded that with a design for a self-closing flood barrier as such, the failure processes should have an individual failure probability of that order of magnitude as indicated in the fault tree in Figure K.3. This can be achieved by selecting components available on the market that contribute to this by having features that reduce the probability of occurrence of certain failure processes, such as a pipe with filter to prevent clogging or adding a heat element to prevent frost. Alternatively, components associated to the failure processes should be tested and designed in such a way that they comply to the failure probability requirements. In conclusion, for a potential follow-up design, a detailed fault tree analysis should be performed with scientifically or statistically supported values for the failure probabilities. In this thesis no further elaboration is done on this.

Furthermore, as a follow up step, increasing the number of closing sections n_{gates} could be considered. This in fact results, on the one hand, in a reduction of the occurrence of some events, such as an obstacle on the barrier. On the other hand, the total failure probability is multiplied by a larger factor, because the barrier has more individually closing gate parts. Reducing the number of closing sections n_{gates}, thereby creating more margin in the failure probability requirement per dike section would be worth looking into as well. Side note however, is that the span of individually closing gate parts will become larger and the likelihood of the aforementioned events actually increases.

L

Other functional design features

L.1. Determination of floater width

In this section the minimum floater width is determined based on Archimedes' principle, using initial assumptions for the geometry of the barrier. Firstly some parameters are defined:

$A_{s,ret}$	[m²]	**=**	cross-sectional area of retaining part of barrier
$A_{s,flt}$	[m²]	=	cross-sectional area of floater part of barrier
A_{dw}	[m²]	=	area of displaced water
B	[kN]	=	buoyancy
W	[kN]	=	weight of barrier
a	[m]	=	width of floater
b	[m]	=	height of floater
t	[m]	=	thickness of plate elements of floater

The assumptions that are done are presented here next to the specific weight of steel and water:

Parameter	**Value**	**Unit**
$A_{s,ret}$	0.07	m²
b	0.35	m
t	0.01	m
γ_w	10.0	kN/m³
γ_s	78.5	kN/m³

The required area of the floater and naturally the width of floater follows from Archimedes' principle:

$$B = W \tag{L.1}$$

$$B = A_{dw} \cdot \gamma_w \tag{L.2}$$

$$W = (A_{s,ret} + A_{s,flt}) \cdot \gamma_s \tag{L.3}$$

Equations L.1, L.2 and L.3 result in:

$$A_{dw} \cdot \gamma_w = (A_{s,ret} + A_{s,flt}) \cdot \gamma_s \tag{L.4}$$

The area of the displaced water is defined by:

$$A_{dw} = a \cdot b \tag{L.5}$$

The cross-sectional area of floater part is defined by:

$$A_{s,flt} = (a \cdot b) - ((a - 2 \cdot t) \cdot (b - 2 \cdot t)) \tag{L.6}$$

Equations L.4, L.5 and L.6 result in:

$$a \cdot b \cdot \gamma_w = (A_{s,ret} + (a \cdot b) - ((a - 2 \cdot t) \cdot (b - 2 \cdot t)) \cdot \gamma_s \tag{L.7}$$

Solving Equation L.7 for a with the assumed parameter A_{ret}, b and t leads to a floater width of:

$$a = 2.8m$$

L.2. Static floating stability

Floating objects have a certain sensitivity to tilting. For the self-closing flood barrier this is not preferable because the sides of the floater may hit the walls of the concrete, leading to possible jamming of the movement. The resistance to tilting is given by a requirement for the metacentric height of the object. The metacentric height is the point of intersection of the axis of symmetry, the z-axis and the action line of the buoyant force in tilted position. The metacentre should be higher than the centre of gravity of the floating structure with respect to the bottom of the structure in order to have static stability. This is because the buoyant force compensates the rotation of a floating stucture with a righting moment. The arm of this righting moment depends on the metacentric height. If a floating structure with a small width is in a tilted position, the metacentric height may be too small and the buoyant force will not cause a sufficiently compensated righting moment to ensure static stability.

In conclusion, the floater of the self-closing flood barrier needs to have sufficient width in order to ensure static floating stability. Below a comparison is shown for a self-closing flood barrier executed with two types of floaters differentiating in their geometrical shape.

$$KG = \frac{\sum_i^n V_i \cdot d_i}{\sum_i^n V_i} \tag{L.8}$$

$$BM = \frac{I_{yy}}{V_{dw}} \tag{L.9}$$

$$I_{yy} = \frac{1}{12} \cdot L \cdot B^3 \tag{L.10}$$

$$KB = \frac{h_{dw}}{2} \tag{L.11}$$

$$h_m = GM \geq 0.5m \tag{L.12}$$

$$GM = KB + BM - KG \tag{L.13}$$

In Figure L.1 tilting of the floating barrier is schematised, in which is shown that the floating stability is ensured.

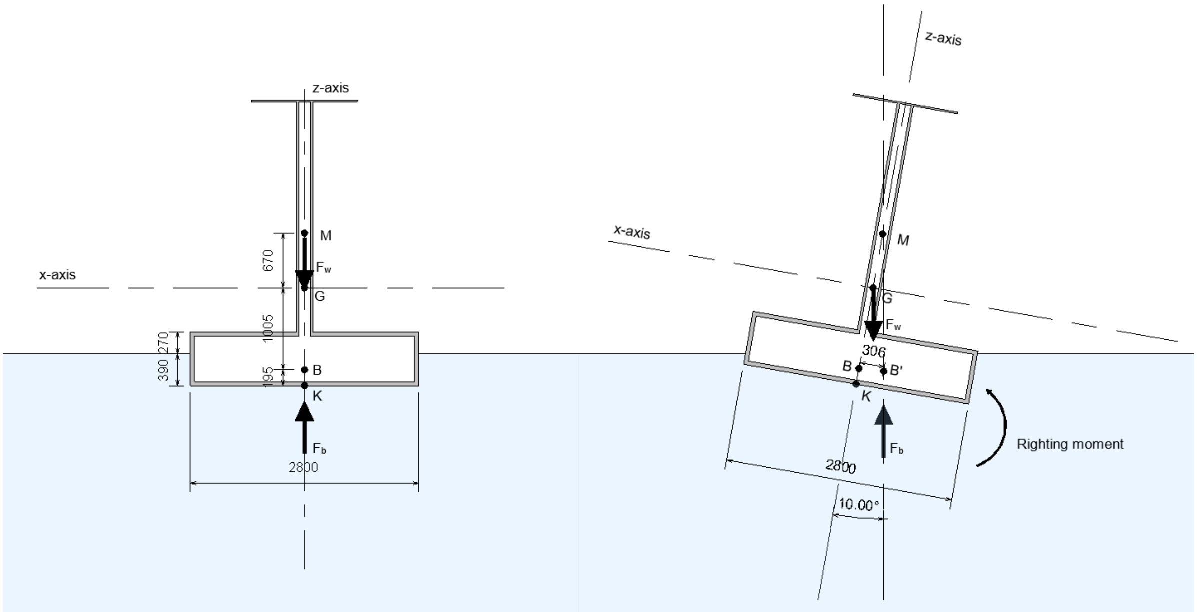


Figure L.1: Schematisation of tilt of floating barrier

In the following tables, the calculation and results are shown for the metacentric height.

Parameter	Value	Unit
A_1	0.07	m
A_2	0.07	m
L	40.0	m
$V_1 = A_1 \cdot L$	2.80	m
$V_2 = A_2 \cdot L$	2.80	m
z_1	2.08	m
z_2	0.33	m
B	2.8	m
h_{dw}	0.39	m
$V_{dw} = B \cdot L \cdot h_{dw}$	43.68	m
KB	0.195	m
BM	1.675	m
KG	1.20	m
GM $= h_m$	0.67	m

Parameter	Value	Unit
A_1	0.07	m
A_2	0.035	m
L	40.0	m
$V_1 = A_1 \cdot L$	2.80	m
$V_2 = A_2 \cdot L$	1.40	m
z_1	2.17	m
z_2	0.38	m
B	1.1	m
h_{dw}	0.75	m
$V_{dw} = B \cdot L \cdot h_{dw}$	33.0	m
KB	0.375	m
BM	0.134	m
KG	1.57	m
GM $= h_m$	-1.06	m

M
Constructibility

This Appendix elaborates on the construction method, the excavation technique, the foundation method, transport and logistics and lastly the construction sequence for the self-closing flood barrier. This will gain insight in the possible failure mechanisms and if and how the structure is constructible.

M.1. Construction method

The construction method is a combination of prefabrication and in-situ. The concrete is casted in-situ, the main structural frame of the steel gate is prefabricated, transported to the construction location and connected to the system on site. Below the justification can be found for the selection of the construction methods.

The concrete chamber which functions as the embedment of the gate and the floatation chamber of the gate is casted in situ. The main reason for this is the water tightness requirement of the structure. Besides this, casting the concrete over the entire span requires less labour, because with prefab concrete elements, connection of the elements is necessary afterwards. Moreover, irrespective of the fact that the area is accessible for construction, prefab elements require transportation to the project location using construction roads connected with Maasstraat. This is costly, but also increases the construction hindrance in the city. The ground water level is also above the bottom level of the foundation. So with prefab concrete elements, constant dewatering for a dry pit is required. With construction in situ, an underwater floor could be chosen.

Considering the steel gate parts, the manufacturing of the main structural frame will be prefabricated at a different location and transported to the construction location. At the construction location, the gate parts will be installed and connected into the system. The gate parts as mentioned here involve the gate leaf and the floater. The steel cover deck on the consoles is considered part of the underground structure. The top cover has the function to cover the gate recess in order to keep the structure out of sight. The cover needs to be placed at last when the steel consoles are mounted with bolts to the concrete walls. The assembly of the the steel consoles will be done on site since the self-closing flood barrier needs to be suspended first partially, because otherwise the width of the recess is too small to suspend the barrier into the concrete structure. The manufacturing of the gate leaf, involving welding of the girders and posts to the skin plate, are preferred to do in a controlled environment to ensure the quality of the welds. Welding is also a laborious procedure which makes it preferable to do it in a controlled environment to have good working conditions.

M.2. Excavation technique

This section describes two excavation techniques suitable for the construction of the self-closing flood barrier after which one technique will be chosen.

Construction pit

A construction pit bounds the excavated area with natural slopes. The ground water table is often lowered beneath the bottom level of the pit with the help of a dewatering system to construct the structure in the dry. A construction pit is generally used at locations with sufficient space. The natural slopes take additional space, because of requirements for the slope stability. Otherwise, the soil on the slopes can mobilise, which could compromise the water retaining function of the construction pit (Molenaar & Voorendt, 2023). An example is given in Figure M.1.

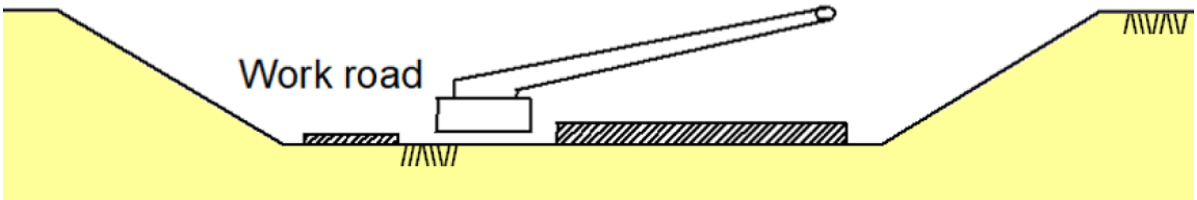


Figure M.1: Example of schematic cross-section of construction pit (Molenaar & Voorendt, 2023)

Cofferdam

A cofferdam is a type of excavation of an area for construction, which consists of a horizontal sealing and vertical walls that retain soil and water, such as sheet piles. The horizontal sealing can be an impermeable soil layer or for example an underwater concrete slab (Molenaar & Voorendt, 2023). The stability of the vertical walls is often ensured by installation anchors on the outside of the wall or installing struts between the walls. If an underwater concrete floor is required as horizontal sealing, because for example no impermeable layers are present in the subsoil, it may be required to install tension piles as well, in order to prevent uplift of the floor. Cofferdams are generally more expensive than construction pits with natural slopes (Molenaar & Voorendt, 2023). However, cofferdams are more suitable in urban areas where space is limited (Molenaar & Voorendt, 2023). Also, a drainage system is not required but optional. Dewatering of the cofferdam, after sealing the excavated pit, can be arranged by pumps. An advantage of the cofferdam is that the temporary structure could be integrated in the permanent structure. For example, sheet pile walls can also be used as piping screens and the underwater concrete floor can be used as permanent floor of the structure. An example is given in Figure M.2.

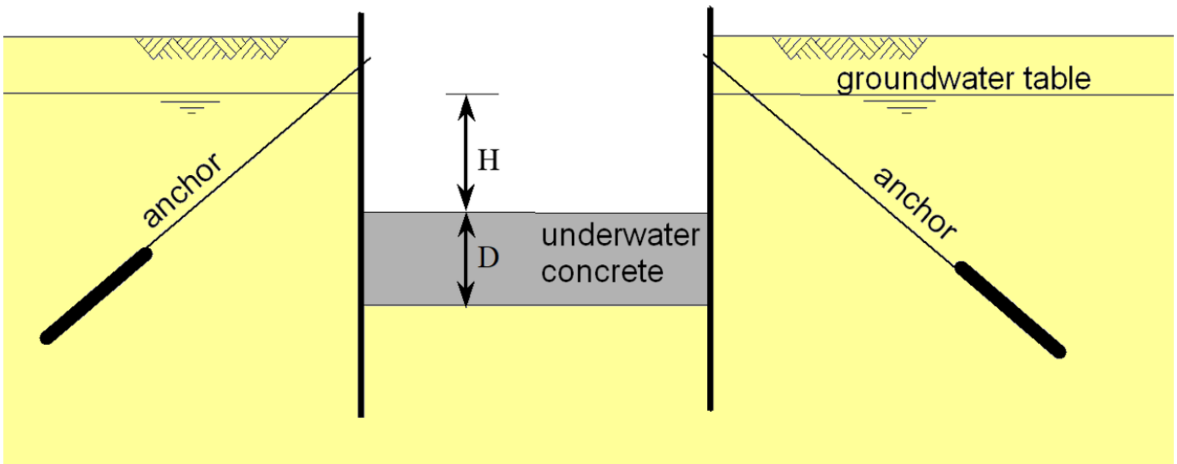


Figure M.2: Example of schematic cross-section of cofferdam (Molenaar & Voorendt, 2023)

Selection excavation technique

A construction pit with natural slopes takes relatively much space which is not suitable in an urban area, because at the land side of the barrier there is not much space between the barrier and the houses of the residents. Also, a construction pit in the area affects the spatial quality too much, related to societal living conditions, since the area also consists of touristic walking routes. Moreover, the location of the barrier is in private gardens, for which it is preferable to minimise the effect of construction hindrance on these private properties, since it will already be affected by construction works. A cofferdam take less space and is thus the more forward option regarding this aspect. Another important downside of a construction pit is lowering of the groundwater table, because this could lead to settlements in the area which is harmful for existing foundations. This is not a problem with a cofferdam. Thus, the characteristics of the cofferdam are more suitable for this project than those of a construction pit. For this reason, a cofferdam is chosen as excavation method, even though it is a more expensive alternative. A cofferdam can be executed with an underwater concrete floor or with the help of drainage. In this project is chosen for an underwater concrete floor, since the floor needs to be casted anyways and tension piles may not be required since the upward ground water pressure is not high.

M.3. Foundation method

For the foundation method there are two main possibilities, namely a shallow foundation or a pile foundation. The structure is embedded into the soil with a depth of approximately 4.75 m, so excavation is already required. It is important to check what the soil conditions are at the foundation depth and below to decide which foundation method to choose. In the Basis of Design in Chapter 3, the boundary conditions related to the soil structure were already covered in which it was pointed out that the soil consists of mainly fine sand (DINOloket, 2022). The CPT graphs for the area are derived from DINOloket (2022). From these CPT graphs it can be seen that at a depth of 4 m and lower below the ground surface level, the soil has sufficient cone resistance (> 5-8 MPA) and thus consists of already load bearing sand layers on which directly can be founded. Furthermore, settlements are unlikely to occur since the weight of the excavated soil is larger than the substituted weight of the structure. For the same reason, uplift in the governing case will not occur since the ground water pressure does not exceed the weight of the structure.

For this reason a pile foundation is not necessary. So the structure will be founded with a shallow foundation. The structure is relatively small in width and because a horizontal sealing for the cofferdam is needed, since no impermeable layers are present, a slab foundation is chosen for this on which both bearing walls of the concrete chamber can transfer the loads. This design choice is also in accordance with the excavation technique in which was already mentioned that an underwater concrete floor has the preference. Since the foundation depth is already 4.75 m below ground level, it is safe to assume that the frost line is above the foundation depth because the minimum depth to prevent freezing of the soil is often 0.6 to 0.8 m below ground level.

M.4. Transport and logistics

As has already been pointed out, the construction site is located in private backyards along the Meuse over a stretch of more than 200 m. This means that a part of the private gardens must be used for construction works, such as excavation, sheet piling and concrete casting. In Figure M.3 it is illustrated in purple on a map of the construction area which part of the private properties needs to be used. For this construction hindrance the residents need to be informed and compensated to restore their gardens after the construction works are finished. Further study on this is outside the scope of this thesis.

In view of reducing the construction hindrance and minimising the affect on the spatial quality during the construction period, there are few aspects to consider. Because of limited space in the area, the construction area will be located solely on the river side of the barrier. This results in closure of the Burgemeester Linders-Promenade for the public. The touristic promenade along the Meuse will be open from the south up to the 'Maasterras'. In this way the touristic area is still partially accessible for the public. This is shown in Figure M.3. Transport and short-term storage for construction materials, structural components and equipment can be stored at a small site of 0.1 ha which is located in the northern corner of the Burgemeester-Linders Promenade which is accessible from the Maasstraat. In

Figure M.3 the location for this site is shown in blue. Construction traffic from the south will be limited on the main road ’Maasstraat’. The reason for this is that this part of the Maasstraat is narrow and involves the accessibility to the city centre where most of the hospitality and retail businesses are located. The construction area is good accessible from the north via the part of the Maasstraat which is a rural road connected to the provincial road N271. This is also shown in Figure M.4 and Figure M.5. Transport via these roads will minimise the construction hindrance.

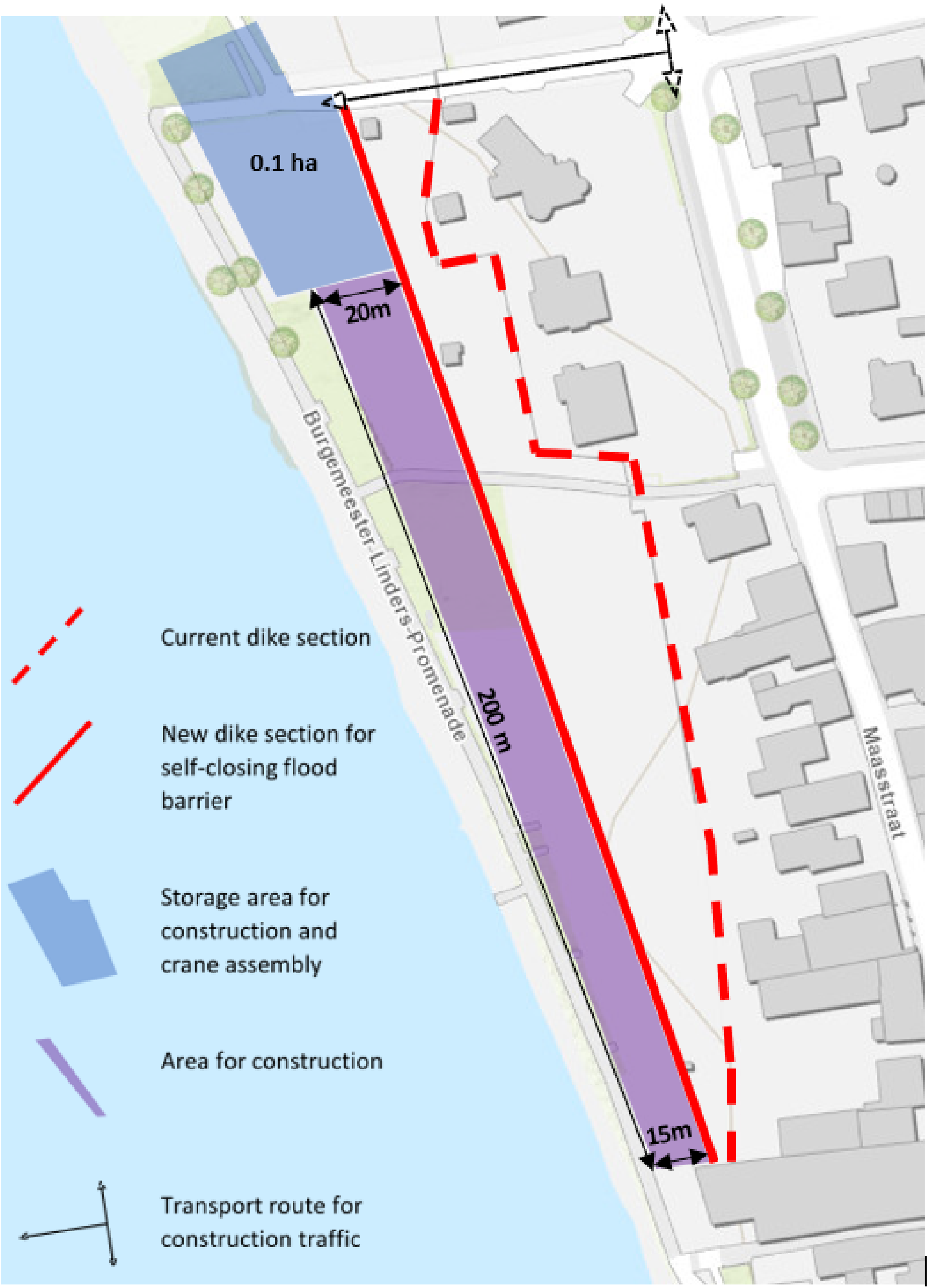


Figure M.3: Map with the construction area indicated

Figure M.4: Logistics map

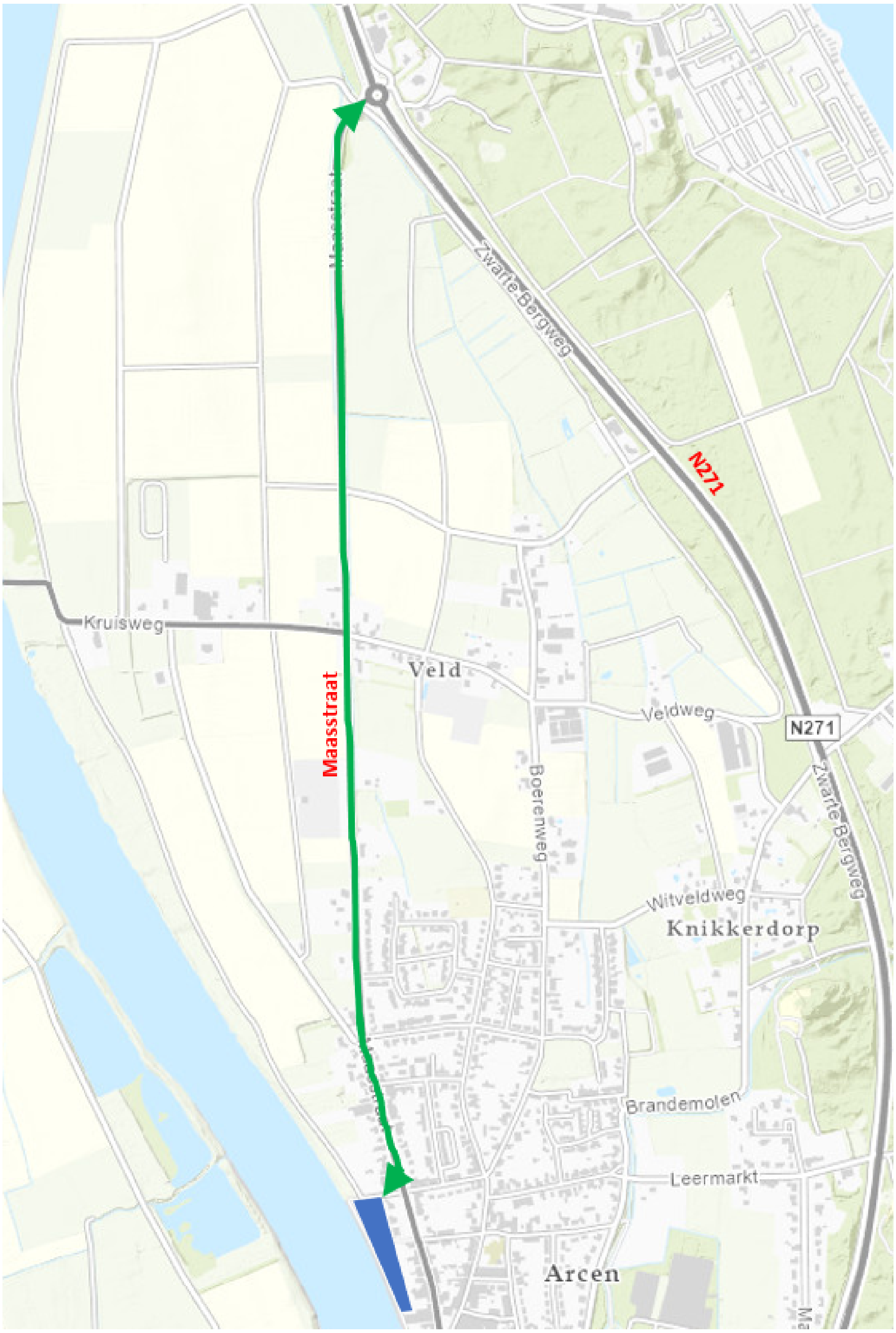
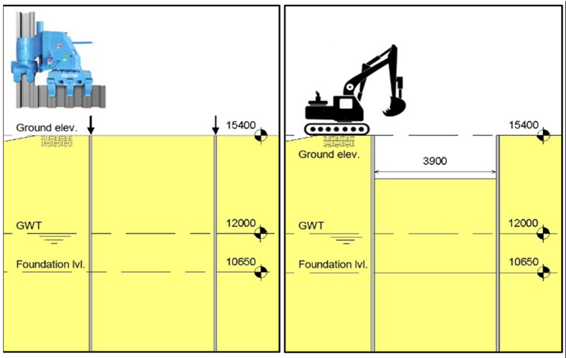


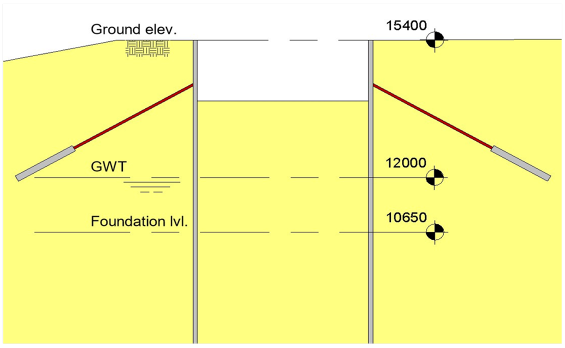
Figure M.5: Map indicating access to area for construction traffic

M.5. Construction sequence



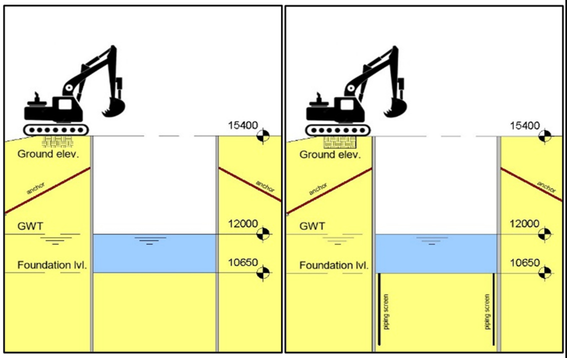
1. Sheet piling and initial excavation

Firstly, sheet pile walls are installed inside the soil at the required embedded depth. These sheet pile walls function as the vertical walls of the cofferdam. Since the construction area is in the proximity of existing buildings consisting of mainly houses, it is preferable to minimise the construction hindrance, by using for example silent pilers that press-in instead of vibrate. When the sheet pile walls are in place, the cofferdam can be excavated to a certain intermediate level. The cofferdam will not be fully excavated because the sheet pile wall need to be supported first in order to ensure their stability.



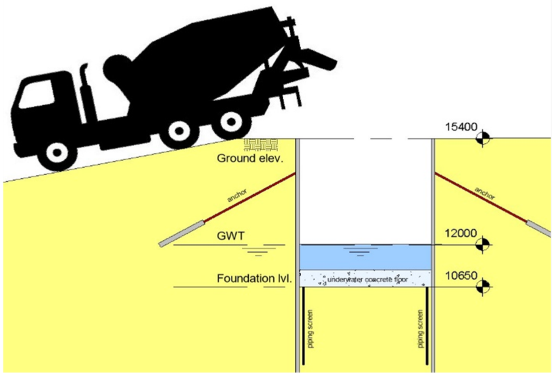
2. Installing grout anchors

The sheet pile walls will be anchored in the soil in order to prevent the walls from being pushed away by the horizontal active soil pressure.



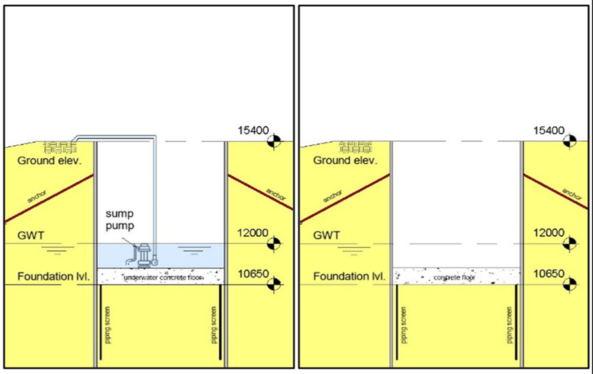
3. Excavation to completion

The cofferdam is now ready to be excavated to its ultimate level. Medium sized excavators are able to excavate to a depth between 5 - 7 m. So excavation can be done from the ground surface. The excavated soil will be used to level the surface around the structure to NAP + 15.4 m. The final excavated level lies below the ground water table, which means that water will flow in to the cofferdam with a water depth of approximately 1.35m. The next step is to place the piping screens into the ground beneath the foundation level where structure will be constructed.



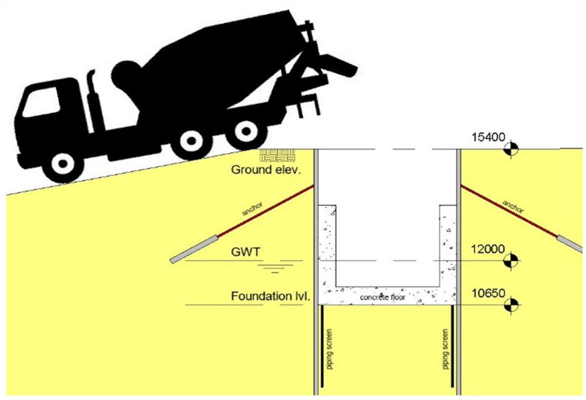
4. Casting of underwater concrete floor

The cofferdam needs to be sealed from the bottom in order to create a dry pit. This is done by casting an underwater concrete floor as a horizontal sealing. Note: tension piles may be needed also, but this will follow from the vertical stability verification. Tentatively, this step is excluded from this construction sequence.



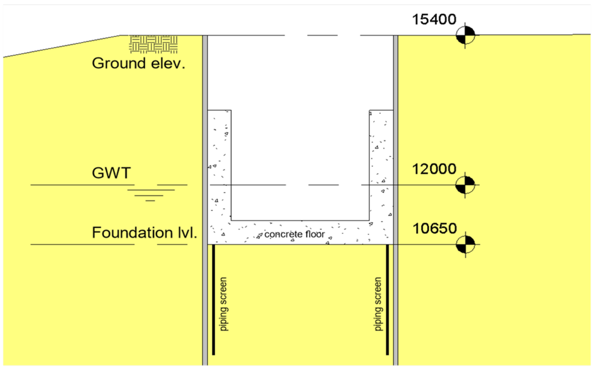
5. Dry pumping cofferdam

When the underwater concrete floor is hardened and the horizontal seal is complete, the cofferdam can be pumped dry by sump pumps.



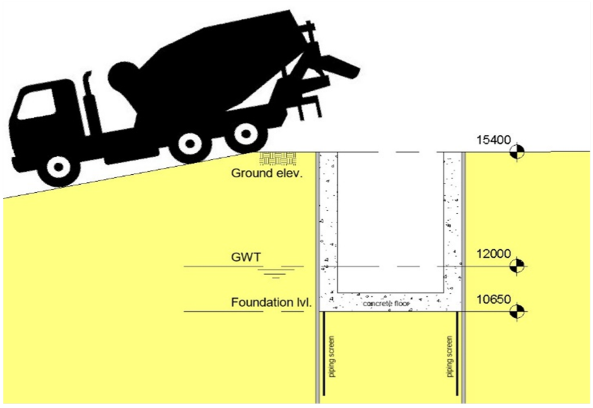
6. Casting of concrete walls

The next step is to cast the concrete walls to create the floatation chamber of the self-closing flood barrier. The underwater concrete floor may be integrated as the floor of the permanent structure. Before the actual casting, the formwork need to be installed in place and the steel reinforcement need to be placed. After concrete hardening and reaching its full strength, the inside of the concrete will be finished for the guidance rollers to ensure minimum friction to the movement of the gate. Also, to the top part of the walls anchor plates will be attached for connecting steel consoles and cover plates in the next step.



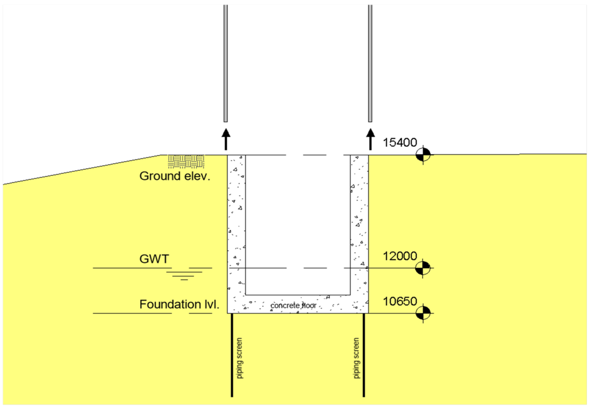
7. Removal of grout anchor

When the concrete walls with partial height are casted and hardened, the grout anchors can be removed, because the sheetpile walls have sufficient support by the concrete walls.



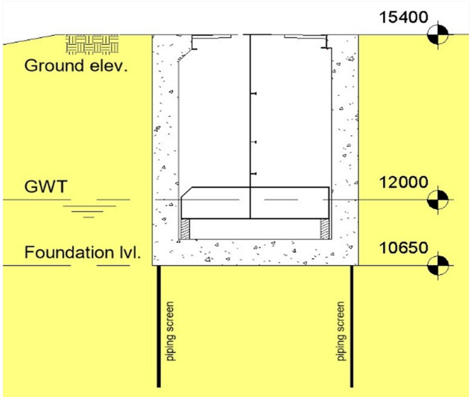
8. Finish casting concrete walls

After removal of the grout anchors, the casting of the concrete walls can be finished so that the walls are casted until the ground level of NAP + 15.4 m.



9. Removal of sheetpile walls

When the top part of the concrete walls are hardened, the sheetpile walls can be removed.



10. Gate suspension and attachment of consoles

After preparing the inside of the concrete, the prefabricated steel gate parts are transported to the construction area and initially hoisted partially into the concrete structure. The reason that they are partially suspended in the concrete structure is needed in order to attach the steel consoles with top cover and seals first to the concrete with bolts to close off the recess. The gate parts will one by one be suspended totally afterwards onto timber blocks.

N

Loads

In this appendix additional elaboration is given for the determination of the loads.

N.1. Horizontal effective soil pressure

N.1.1. Theoretical background

The total horizontal soil pressure consists of the horizontal effective soil pressure and the groundwater pressure. According to Pascal's law, water pressure is equal in all directions (Elger, Williams, & Crowe, 2013), but this does not apply to soil pressure. Thus, both require separate consideration. The water pressure is elaborated further as part of the variable loads. The horizontal effective soil pressure is linearly related to the vertical effective soil pressure σ_v' by a constant factor K . The vertical effective soil pressure in turn is determined by the following relation.

$$\sigma_v' = \sum_{i=1}^{n} \gamma_{d,i} \cdot d_i + \sum_{j=1}^{m} \gamma_{n,j} \cdot d_j - p \tag{N.1}$$

, in which:

σ_v'	[kN/m²]	**=**	vertical inter-granular stress (= effective pressure)
$\gamma_{d,i}$	[kN/m³]	=	dry volumetric weight of soil layer i
$\gamma_{n,j}$	[kN/m³]	=	wet volumetric weight of soil layer j
d_i	[m]	=	thickness of soil layer i above the considered plane
n	[-]	=	number of dry layers above the considered plane
m	[-]	=	number of wet layers above the considered plane
p	[kN/m³]	=	water pressure in the considered plane

In summary, the vertical effective soil pressure is determined by multiplying the specific weight of the soil type in each layer with the layer thickness and adding up each stress value per layer until the stress in the considered plane is obtained, which is at the bottom of the structure subjected to the soil pressure. For wet layers, where groundwater is present, the volumetric weight of water needs to be subtracted of the specific weight of the soil.

The soil along the span of the barrier varies, which means that the soil parameters are different for several locations along the span of the barrier. From DINOloket (2022), the central gateway to data and information of the Dutch subsoil, there is data available of four relevant locations along the span of the barrier. The locations are shown in Figure N.1. In Appendix D, soil profiles are reconstructed from CPT graphs of these four locations and with the help of the Manual Hydraulic Structures of Voorendt

(2023) for the classification of the soil types.

Figure N.1: Locations of retrieved soil data (DINOloket, 2022)

The horizontal effective soil pressure is calculated according to Rankine's theory. Rankine's theory is based on finding the upper and lower limit for the horizontal effective soil pressure, which corresponds to two types of deformation, active and passive deformation (Voorendt, 2023). The real value for the active and passive horizontal soil pressure lies between these two limit values. This means that this theory uses a conservative approach to determine the horizontal soil pressure, because it calculates the maximum possible value for the active and passive horizontal soil pressure. This conservative approach is preferable for safety purposes. The relation of Rankine's theory for active slip planes is given below.

$$\sigma'_{h,min} = K_a \cdot \sigma'_v - 2c\sqrt{K_a} \tag{N.2}$$

$$\sigma'_{h,max} = K_p \cdot \sigma'_v - 2c\sqrt{K_p} \tag{N.3}$$

with:

$$K_a = \frac{1 - sin\phi'}{1 + sin\phi'} \tag{N.4}$$

$$K_p = \frac{1 + sin\phi'}{1 - sin\phi'} \tag{N.5}$$

, where:

For sand layers, the soil is non-cohesive so $c = 0$. The weighted internal friction angle for the layered soil is determined by:

σ_h'	[kN/m²]	**=**	horizontal effective soil pressure
K_a	[-]	=	coefficient of active soil pressure
K_p	[-]	=	coefficient of passive soil pressure
c	[-]	=	cohesion
ϕ'	[°]	=	weighted angle of internal friction of soil layers

$$\phi' = \frac{\sum_{i=1}^{n} \phi_i \cdot h_i \cdot X_i}{\sum_{i=1}^{n} h_i \cdot X_i} \tag{N.6}$$

, in which:

ϕ'	[kN/m²]	**=**	weighted angle of internal friction of soil layers
ϕ_i	[kN/m³]	=	angle of internal friction of soil layer i
h_i	[kN/m³]	=	thickness of soil layer i
X_i	[m]	=	distance between the centre of layer i to the influence depth D_{max}
D_{max}	[-]	=	influence depth (= 6.4 m + NAP)
n	[-]	=	number of layers between construction level and the influence depth

The influence depth D_{max} is chosen to be at the lowest depth that the CPT graphs in Appendix D have data for, which is at 6.4 m + NAP. For one location the CPT graph had no data below 7.7 m + NAP, but for this location it is assumed that the soil layer is the same to the depth of 6.4 m + NAP. The difference is insignificant to the end result. With the help of the reconstructed soil profiles in Appendix D and the aforementioned relation, the weighted angle of internal friction is calculated for the four locations along the span of the barrier which are:

Parameter	Value	Unit
ϕ_1'	30	[°]
ϕ_2'	30	[°]
ϕ_3'	29	[°]
ϕ_4'	22	[°]
Averaged internal friction and coefficient K		
ϕ'	27.9	[°]
K_a	0.36	[-]
K_p	2.76	[-]

The average of the values for the weighted internal friction of every location is used to calculate factor K_a and K_p.

N.1.2. Resulting pressure diagrams of soil stress

From the reconstructed soil profiles, the soil pressure diagrams over the height of the barrier are created for each of the four locations along the barrier with the relations given in the previous section for σ_v' and σ_h'. The diagrams are one by one presented below.

Pressure diagram location 1

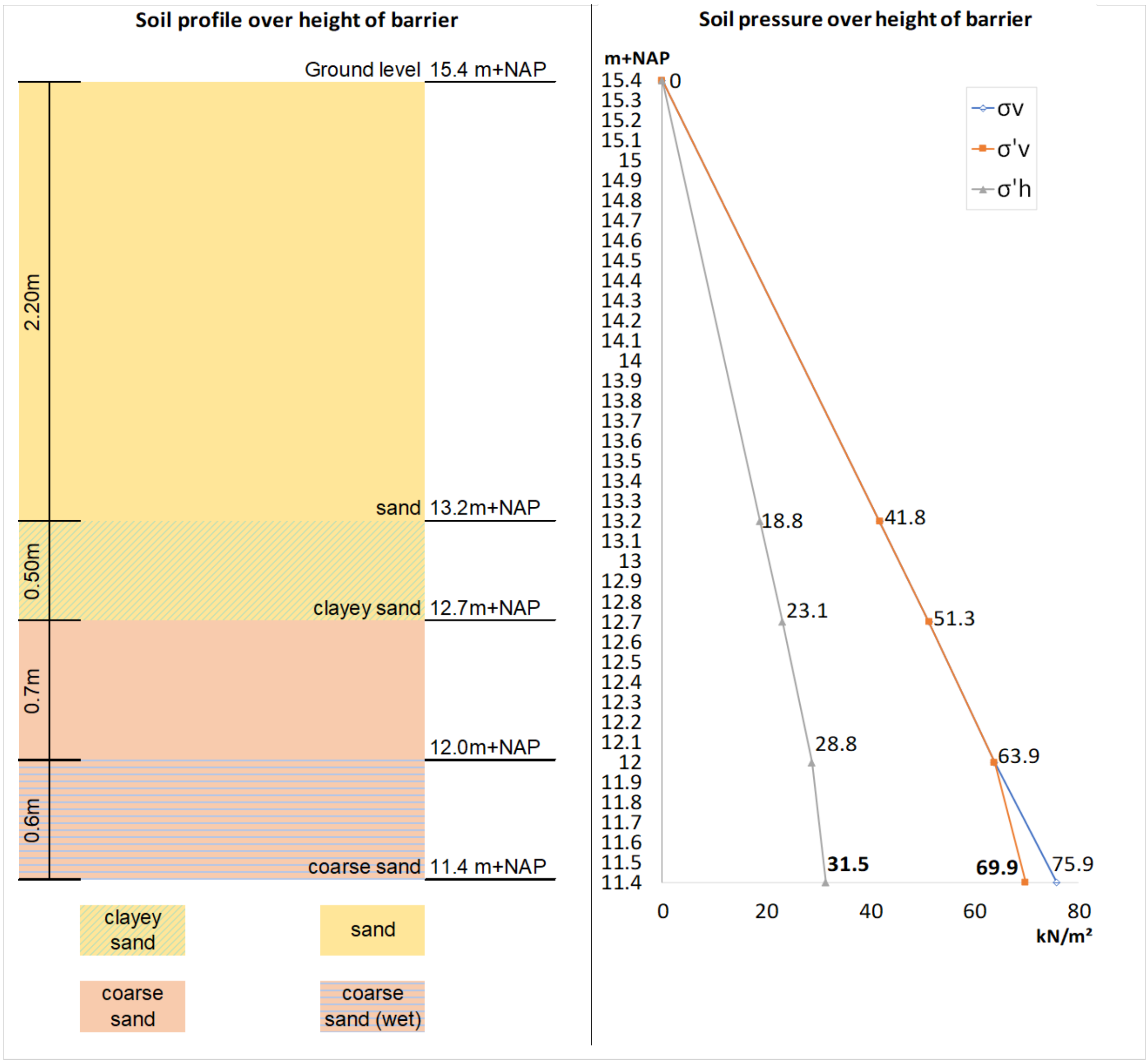


Figure N.2: Soil pressure diagram for location 1 (pressure in kN/m^2 on horizontal axis; level in m from construction level to ground level on vertical axis)

Pressure diagram location 2

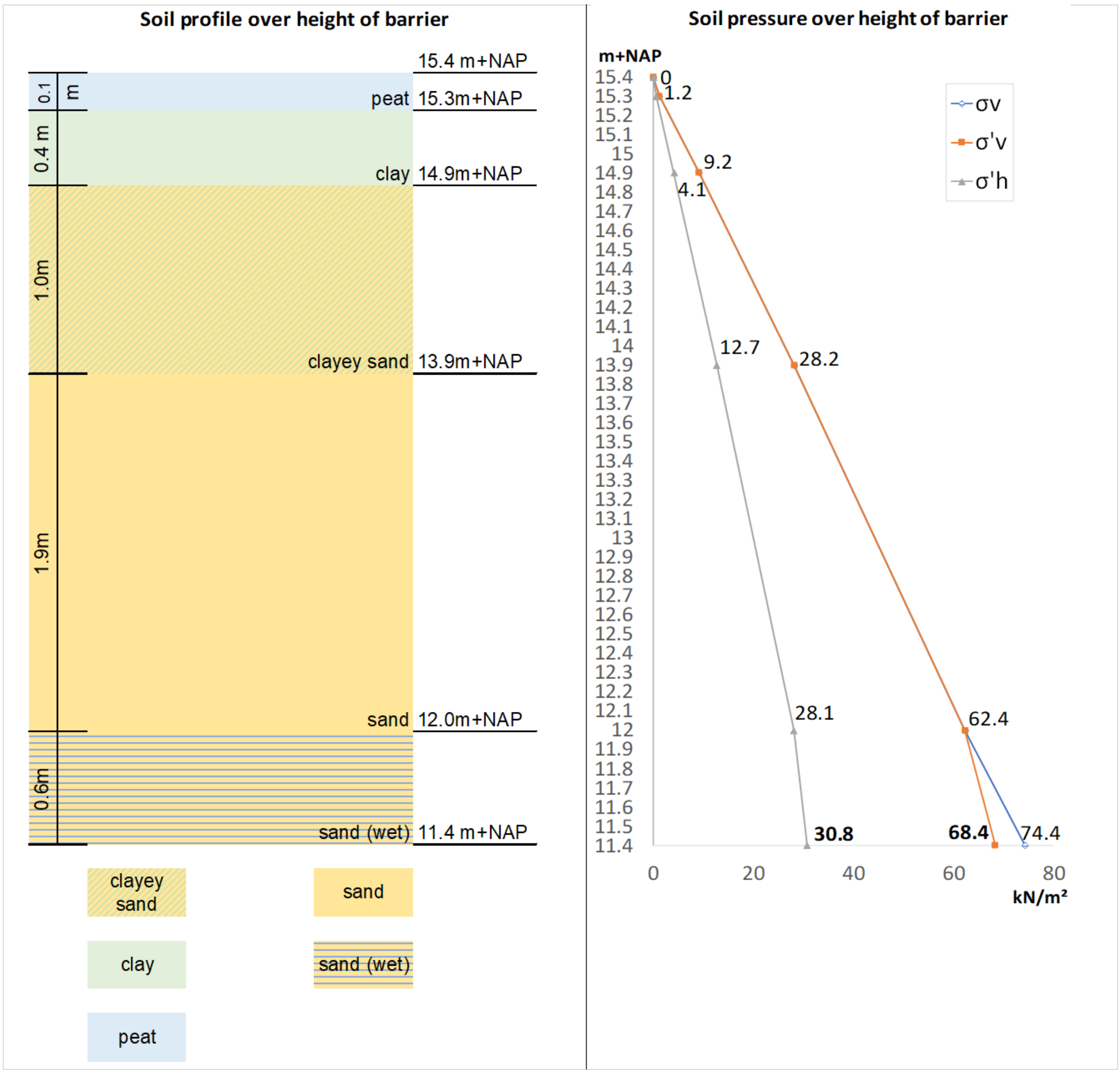


Figure N.3: Soil pressure diagram for location 2 (pressure in kN/m^2 on horizontal axis; level in m from construction level to ground level on vertical axis)

Pressure diagram location 3

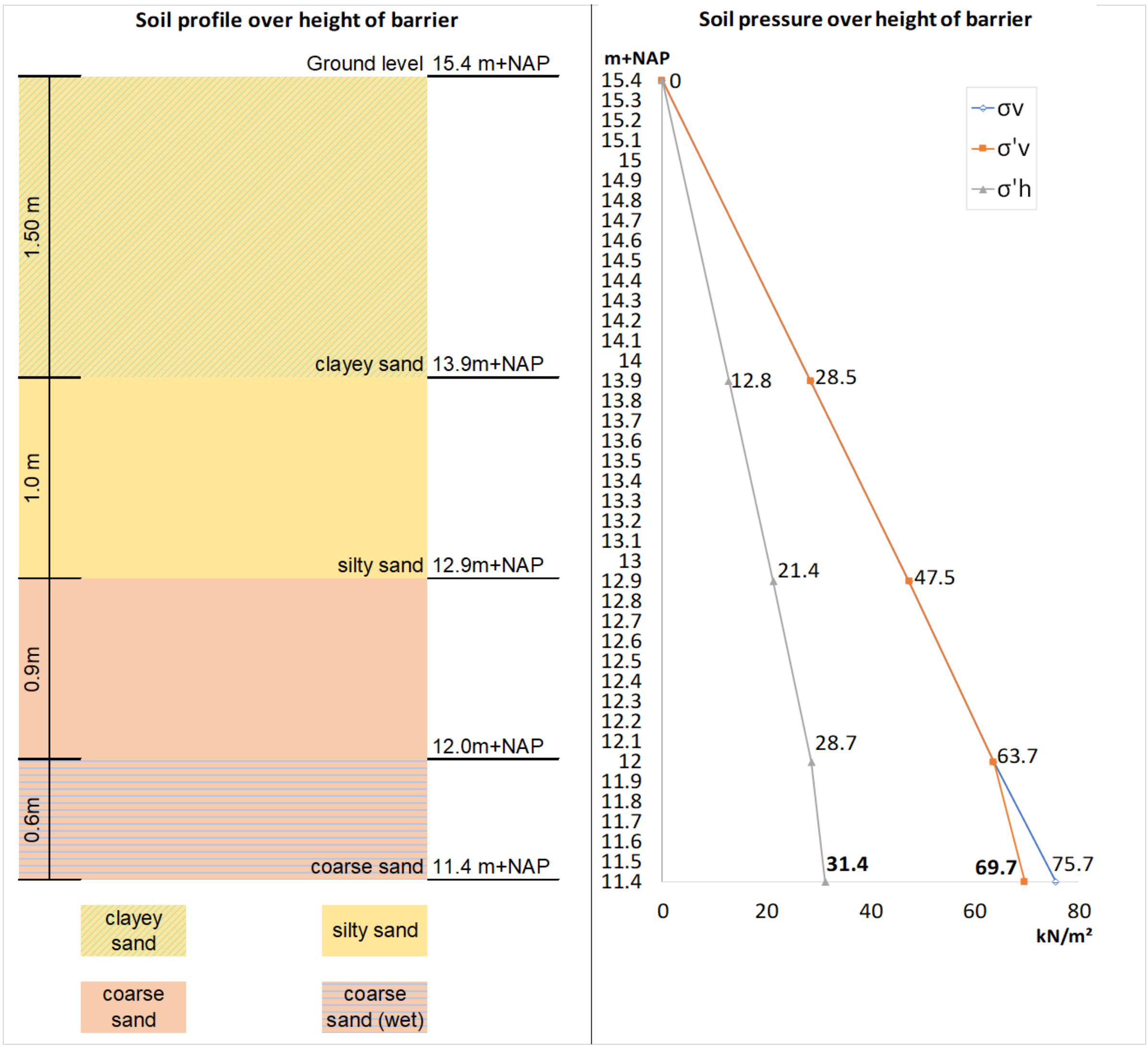


Figure N.4: Soil pressure diagram for location 3 (pressure in kN/m^2 on horizontal axis; level in m from construction level to ground level on vertical axis)

Pressure diagram location 4

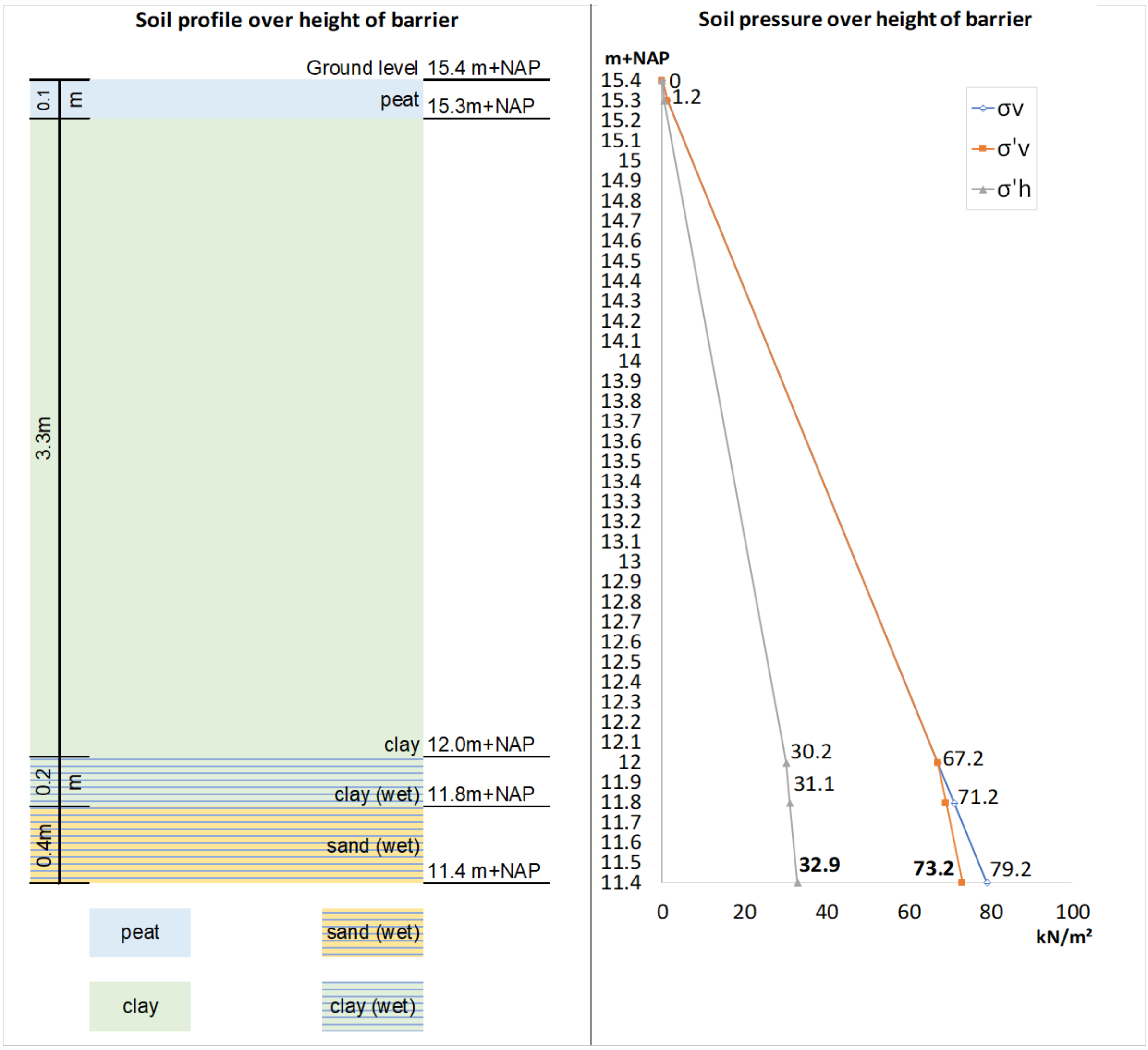


Figure N.5: Soil pressure diagram for location 4 (pressure in kN/m^2 on horizontal axis; level in m from construction level to ground level on vertical axis)

N.1.3. Overarching effective pressure diagram

Examining the results in the previous section, it appears that the diagrams are rather homogeneous and for this reason the horizontal effective soil pressure is assumed to be increasing linearly constant over the height. The results for the average overarching effective soil pressure shown in the diagram in Figure N.6. Thus, this holds for all locations along the entire span, since the differences between the locations are minimal.

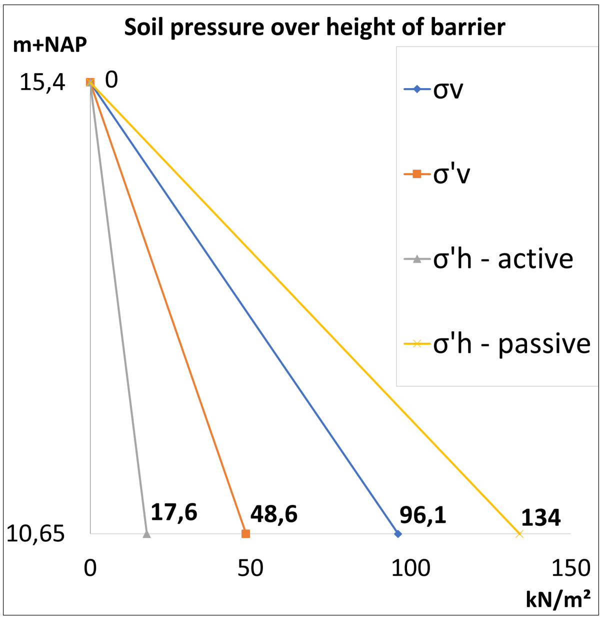


Figure N.6: Overarching effective soil pressure diagram for entire span (pressure in kN/m^2 on horizontal axis; level in m from construction level to ground level on vertical axis)

In Figure N.7 the active and the passive horizontal soil pressure are visualised as calculated:

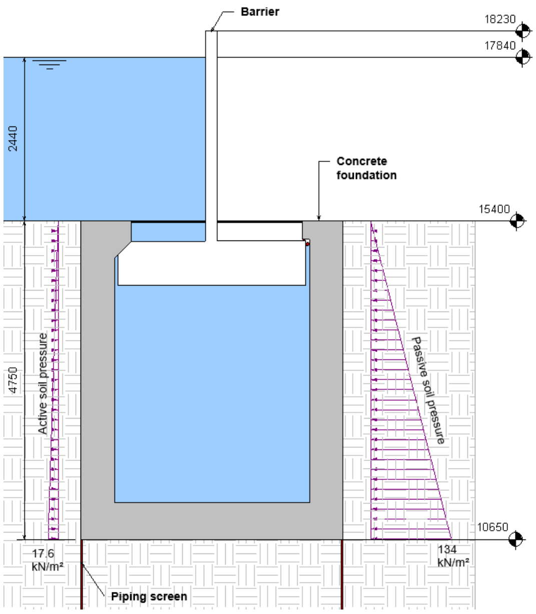


Figure N.7: Acting active and passive horizontal soil pressure

N.2. Variable loads: high water hydraulic loads

The high water hydraulic load consists of two contributions, the first being the hydrostatic pressure on the structure (both the gate and the concrete) resulting from the still design water level to be retained in case of a high water event and the second being the static contribution of wave loads.

1. Hydrostatic pressure still water level
The hydrostatic pressure is linearly distributed with the following relation (Elger, Williams, & Crowe, 2013):

$$P(z) = \rho_w \cdot g \cdot z \tag{N.7}$$

, in which:

$P(z)$	[kN/m²]	=	hydrostatic pressure
ρ_w	[kg/m³]	=	density of water
g	[m/s³]	=	gravitational acceleration
z	[m]	=	depth of considered plane with respect to water surface level

The design still water level is at NAP + 17.84 m. The ground level is at NAP + 15.4 m. This means that the height of the water column to be retained by the gate of the self-closing flood barrier is 2.44 m. The bottom of the concrete structure is at NAP + 10.65 m, resulting in a height of 4.75 m for the concrete structure. With these levels and Equation N.7, the resulting hydrostatic pressure working on the entire structure are calculated and summarised in Figure N.8.

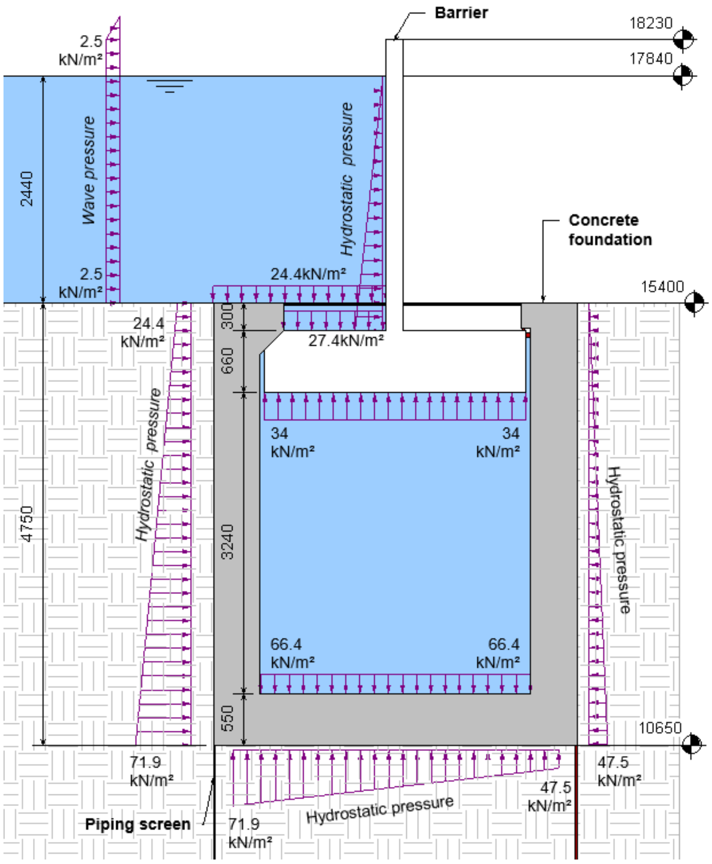


Figure N.8: Hydraulic loads acting on structure

2. Static pressure of wind waves
The self-closing flood barrier is subjected to loads by non-breaking waves. The significant design wave height as calculated with Hydra-NL as part of the hydraulic load level is $H_{M0} = 0.25m$. This is only 10% of the water depth ($= 2.44m$) in front of the structure during the high water event. The loads are therefore relatively low with respect to the hydrostatic pressure. For this reason, the wave pressure will be determined conservatively with a rule of thumb calculating the upper boundary of the wave load acting as a stationary load based on the relation for hydrostatic pressure. This is not based on the linear wave theory. The calculation is shown below and the visualisation in a diagram was already displayed in Figure N.8.

Parameter	**Value**	**Unit**
ρ_w	$1.00 \cdot 10^3$	kg/m³
g	9.81	m/s²
Top level wave	18.1	m+NAP
H_{m0} (wave height)	0.25	m
water depth in front of structure	2.44	m
z_1	0.25	m
$z_2 = 0.25 + 2.44$	2.69	m
$P(z = 0.25m)$	$2.5 \cdot 10^3$	N/m²
$P(z = 2.69m)$	$2.5 \cdot 10^3$	N/m²
$\mathbf{P(z = 0.25m)}$	**2.5**	**kN/m²**
$\mathbf{P(z = 2.69m)}$	**2.5**	**kN/m²**

The wave pressure is translated to a wave force per unit m span width, which is added to the hydrostatic pressure corresponding to the still water level, calculated with the following relation:

$$F_{max} = \frac{1}{2} \cdot \rho_w \cdot g \cdot H_{m0}^2 + d \cdot \rho \cdot g \cdot H_{m0} \quad \text{(N.8)}$$

, in which:

F_{max}	[kN/m]	**=**	maximum wave force per unit m span width
ρ_w	[kg/m³]	=	density of water
g	[m/s³]	=	gravitational acceleration
H_{m0}	[m]	=	significant wave height
d	[m]	=	water depth in front of barrier

Parameter	**Value**	**Unit**
ρ_w	$1.00 \cdot 10^3$	kg/m³
g	9.81	m/s²
H_{m0}	0.25	m
d	2.44	m
$\mathbf{F_{max}}$	**6.3**	**kN/m**

Upward ground water pressure
The upward ground water pressure is again calculated with Equation N.7 and is relevant in case 2 with the non dominant water level during a high water event. The exact signaling water level is not determined. Tentatively, the design water level for the upward ground water pressure will be assumed to be at NAP + 13.65 m. This means that in the governing case, which means an empty floatation chamber and a water level at NAP + 13.65 m, the ground water pressure at the bottom of the foundation (NAP + 10.65 m) will be 30.0 kN/m². The width of the entire structure is 3.90 m. This means an upward force per unit span length of 117 kN/m.

Parameter	**Value**	**Unit**
ρ_w	$1.00 \cdot 10^3$	kg/m³
g	9.81	m/s²
Groundwater table	13.65	m+NAP
Considered plane (bottom foundation)	10.65	m+NAP
$z = 13.65m - 10.65m$	3.00	m
Upward water pressure		
$P(z = 3.00m)$	$30.0 \cdot 10^3$	N/m²
$\mathbf{P(z = 3.00m)}$	**30.0**	**kN/m²**
Upward force per unit span length		
b (total cross-sectional width of barrier)	3.9	m
$\mathbf{q_{up}}$	**117**	**kN/m**

O
Stability verification

In this section the results of the stability verifications are presented, involving horizontal, vertical, rotational stability, uplift and piping.

The stability verification involves the situation of a high water event with flood consequences. This means that both, a verification based on the Eurocode and the Dutch Water Act need to be performed. However, the Design guide Hydraulic Structures states that the verification based on the Dutch Water Act is not required, if consequence class CC3 is applied, the fixed failure probability estimate (ω) for structural failure is used with respect to the other failure mechanisms, a length effect factor of $N = 3$ is used and a verification is done using NEN-EN 1990 6.10b for the loads (if applicable). The reason for this is that in such a case, the Eurocode will always be governing. For the stability, the self-weight is not over 80% of the total load, which means that the hydraulic load is dominant and thus only formula NEN-EN 1990 6.10b ($E_{d,b}$) will be used. But this also means that the verification based on the Dutch Water Act is not required in this case as is described before. NEN-EN 1990 6.10a does not need to be considered. Below the acting loads important for the stability are presented in Figure O.1 and below that the determination.

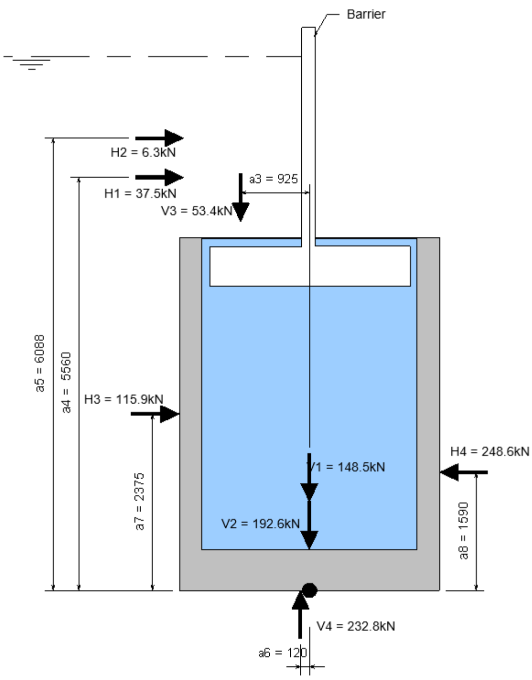


Figure O.1: working loads and lever arms for stability verifications

Parameter	**Value**	**Unit**
Factors		
k_{FI} (factor for upgrade to consequence class CC3)	1.1	[-]
γ_G (partial factor permanent load; favourable)	0.90	[-]
ξ (reduction factor for unfavourable self-weight)	0.89	[-]
Characteristic loads		
Permanent loads (self-weight)		
G_k	168.6	kN
Hydraulic loads		
$S_{d,1}$ (hydrostatic pressure force on gate) = $\frac{1}{2} \cdot P \cdot d = \frac{1}{2} \cdot 27.4 \cdot 2.74$	37.5	kN
$S_{d,2}$ (wave force on gate)	6.3	kN
$S_{d,3}$ (vertical water force inside chamber) = $66.4 \cdot 2.9$	192.6	kN
$S_{d,4}$ (vertical water pressure on top of chamber (riverside) = $24 \cdot 1.95$	53.4	kN
$S_{d,5}$ (upward ground water pressure = $\frac{B}{2} \cdot (P1+P2) = \frac{3.9}{2} \cdot (71.9+47.5)$	-232.8	kN
$S_{d,6}$ (Nett hydrostatic pressure force on concrete) = $P \cdot H = 24.4 \cdot 4.75$	115.9	kN
$S_{d,7}$ (Nett hor. soil pressure force on gate) = TBD in Section O.2	-	kN
Vertical design loads		
$V_{1,d} = k_{FI} \cdot \gamma_G \cdot \xi \cdot G_k = 1.1 \cdot 0.90 \cdot 0.89 \cdot 168.6$	148.5	kN
$V_{2,d} = S_{d,3}$	192.6	kN
$V_{3,d} = S_{d,4}$	53.4	kN
$V_{4,d} = S_{d,5}$	-232.8	kN
$\sum \mathbf{V} = \mathbf{V_{1,d}} + \mathbf{V_{2,d}} + \mathbf{V_{3,d}} + V_{4,d}$	**161.7**	**kN**
Horizontal design loads		
$H_{1,d} = S_{d,1}$	29.3	kN
$H_{2,d} = S_{d,2}$	6.3	kN
$H_{3,d} = S_{d,6}$	115.9	kN
$H_{4,d} = S_{d,7}$	TBD	kN
$\sum \mathbf{H} = \mathbf{H_{1,d}} + \mathbf{H_{2,d}}$	**35.6**	**kN**

O.1. Vertical stability

The vertical effective soil stress required to resist the acting loads $\sigma_{k,max}$, should not exceed the maximum bearing capacity of the soil p'_{max}, otherwise the soil will collapse:

$\sigma_{k,max} < p'_{max}$

The collapse of the soil with slip planes is illustrated in Figure O.3.

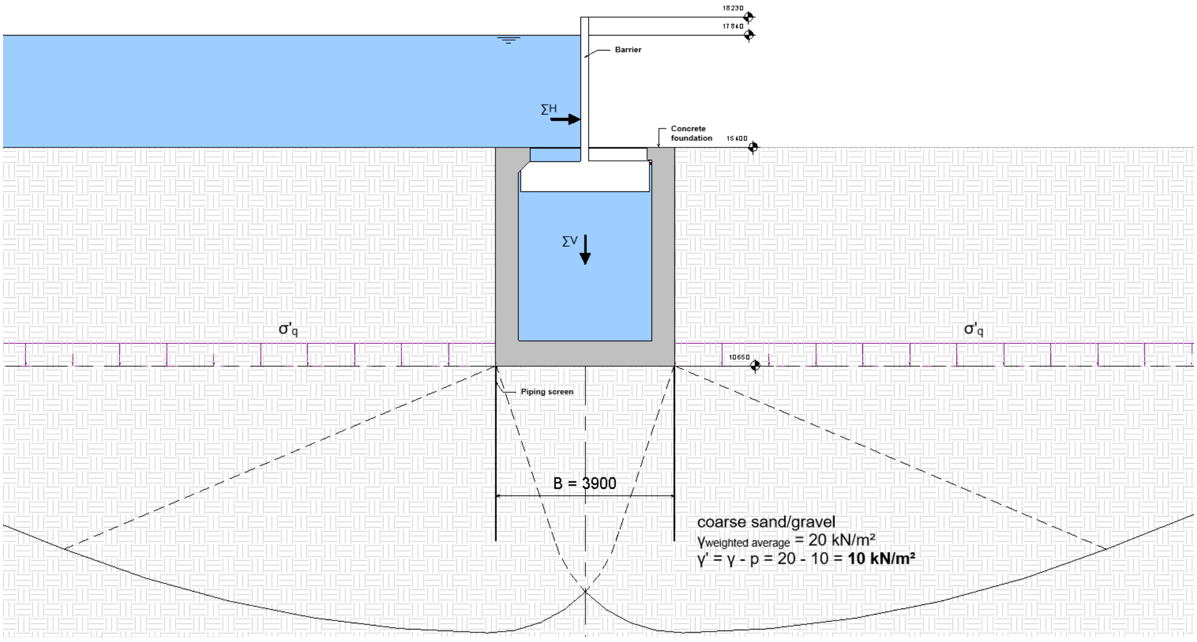


Figure O.2: Failure mechanism of soil collapse under the structure based on Prandtl's method of theoretical slip planes

O.1.1. Vertical effective soil stress

The vertical effective soil stress is determined with:

$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{b \cdot L} + \frac{\sum M}{\frac{1}{6} \cdot L \cdot b^2} \tag{O.1}$$

The vertical effective soil stress per unit span length is determined with:

$$\sigma_{k,max} = \frac{F}{b} + \frac{M}{W} = \frac{\sum V}{b} + \frac{\sum M}{\frac{1}{6} \cdot b^2} \tag{O.2}$$

The sum of the moments is calculated by adding all the individual moments exerted by the individual forces and the corresponding lever arm with respect to the overturning point which is at the the middle of the bottom of the structure. The equation to determine the sum of the moments is given below (note the minus sign for $V_{3,d}$).

$$\sum M = V_{1,d} \cdot a_1 + V_{2,d} \cdot a_2 - V_{3,d} \cdot a_3 + V_{4,d} \cdot a_6 + H_{1,d} \cdot a_4 + H_{2,d} \cdot a_5 + H_{3,d} \cdot a_7 - H_{4,d} \cdot a_8 \tag{O.3}$$

With Equation O.2 the vertical effective soil is calculated. The results are given below.

Force $V_{i,d}$ or $H_{i,d}$			Lever arm a_i			Moment M_i = Force × a_i		
$V_{1,d} =$	148.5	kN	$a_1 =$	0.00	m	$M_{1,d} =$	0.00	kNm
$V_{2,d} =$	192.6	kN	$a_2 =$	0.00	m	$M_{2,d} =$	0.00	kNm
$V_{3,d} =$	53.4	kN	$a_3 =$	0.925	m	$M_{3,d} =$	−55.0	kNm
$H_{1,d} =$	29.3	kN	$a_4 =$	4.81	m	$M_{4,d} =$	201	kNm
$H_{2,d} =$	6.3	kN	$a_5 =$	5.35	m	$M_{5,d} =$	38	kNm
$V_{4,d} =$	233	kN	$a_6 =$	0.12	m	$M_{6,d} =$	28	kNm
$H_{3,d} =$	115.9	kN	$a_7 =$	2.38	m	$M_{7,d} =$	275	kNm
$H_{4,d} =$	−248.6	kN	$a_8 =$	1.59	m	$M_{8,d} =$	−395	kNm
						$\sum \mathbf{M} =$	**97.1**	**kNm**

Parameter	Value	Unit
$\sum M$	97.1	kNm
$\sum V$	161.7	kN
b	3.90	m
$\sigma_{k,max}$	**80**	**kN/m²**

O.1.2. Vertical soil bearing capacity

The soil bearing capacity is provided by the soil underneath the self-closing flood barrier. The soil underneath the structure consists of either (coarse) sand or gravel, which have high permeability and behave as drained soil. The soil bearing capacity in drained materials is expressed by:

$$p'_{max,drained} = c' \cdot N_c \cdot s_c \cdot i_c + \sigma'_q \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \tag{O.4}$$

, in which the first term consists of factors denoted with a 'c' which indicates the contribution of cohesion and the second term consists of factors denoted with a 'q' indicating the contribution of effective surcharge pressure, which means loading on the soil surrounding the bottom of the structure. The last and third term consists of factors denoted with γ indicating the contribution of the specific weight of the soil underneath the structure.

Contributing terms in $p'_{max,drained}$

Since the soil consists of sand, cohesion is $\underline{c' = 0}$, so the contributions with respect to cohesion are further omitted. The effective surcharge pressure σ'_q is in this case the weight of the soil next to the self-closing flood barrier acting as a load on the soil underneath the structure. The effective soil stress is derived from the four representing reconstructed soil profiles and pressure diagrams in Appendix N. The lowest stress value for the effective vertical soil stress (σ'_v) of these four diagrams at the construction level NAP + 10.65 m will be taken for the contribution of surcharge pressure. This is equal to $\underline{\sigma'_v = 88kN/m^2}$. The specific weight γ' of the layered soil below the concrete structure needs to be calculated by weighted averaging. However, since the soil underneath the foundation only consists of (coarse) sand and gravel, the specific weights are rather homogeneous with a value of $\gamma_i = 20kN/m^3$. These layers are saturated, so the effective specific weight for all layers is $\gamma'_i = \gamma_i - p = 20 - 10 = 10kN/m^3$. This means that the weighted effective volumetric weight of the soil is also $\underline{\gamma' = 10kN/m^3}$ which is multiplied with B, the width of the structure in expression O.4.

In summary , expression O.4 can be simplified to the following expression, because there is no cohesion, $c = 0$:

$$p'_{max,drained} = \sigma'_q \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma \tag{O.5}$$

with the following parameters:

Parameter	Value	Unit
$\sigma_q^{'}$	88	kN/m²
$\gamma^{'}$	10	kN/m²
B	3.90	m

Factors N_X, s_X and i_X in $p_{max,drained}^{'}$

Main bearing capacity factor N_X
This paragraph elaborates on the remaining factors used in expression O.4. The main bearing capacity factors N_X are dependent of the weighted angle of internal friction for the layered soil underneath the structure. The layered soil is rather homogeneous and the angles of internal friction for the soil layers are mostly $\phi_i = 32.5°$, so the weighted angle of internal friction is also $\phi^{'} = 32.5°$. Determined with a standard table and graph in figure 26-6 from the Manual Hydraulic Structures (M. Voorendt, 2023), the factors corresponding to the $\phi^{'} = 32.5°$ are $N_q = 24.6$ and $N_\gamma = 21.9$.

φ'	N_c	N_q	N_γ
0,0	5,0	1,0	0,0
5,0	6,5	1,6	0,1
10,0	8,3	2,5	0,3
15,0	11,0	3,9	1,1
20,0	14,8	6,4	2,7
22,5	17,5	8,2	4,1
25,0	20,7	10,7	6,3
27,5	24,9	13,9	9,5
30,0	30,1	18,4	14,4
32,5	37,0	24,6	21,9
35,0	46,1	33,3	33,7
37,5	58,4	45,8	52,5
40,0	75,3	64,2	83,3
42,5	99,2	91,9	135,3

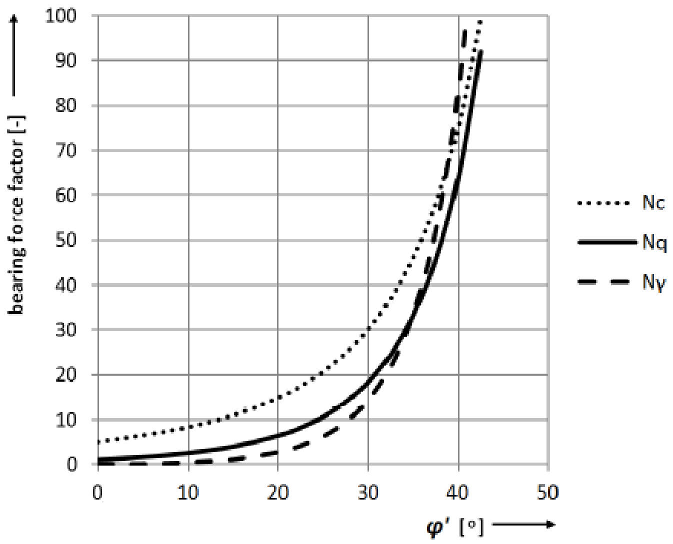


Figure O.3: Bearing force factors as function of the angle of internal friction (Manual Hydraulic Structures, 2023)

Shape factor s_X
The shape factors bring the effect of 3D-slip bodies into account. It depends on the shape of the foundation floor involving the ratio between the width (B) and span length (L), $\frac{B}{L}$. The shape factors can be calculated with:

$$s_q = 1 + \frac{B}{L} \cdot \sin\phi^{'} \tag{O.6}$$

$$s_\gamma = 1 - 0.3 \cdot \frac{B}{L} \tag{O.7}$$

Factor for inclined direction of resultant load i_X
If the horizontal acting load on the structure is parallel to the width, the factors to deal with an inclined direction of the resultant load are calculated with ($c^{'} = 0$):

Parameter	**Value**	**Unit**
B	3.90	m
L	40.0	m
ϕ'	32.5	°
s_q	1.05	[-]
s_γ	0.97	[-]

$$i_q = \left(1 - \frac{0.70 \cdot H}{V + A \cdot c' \cdot \cot\phi'}\right)^3 = \left(1 - \frac{0.70 \cdot H}{V}\right)^3 \tag{O.8}$$

$$i_\gamma = \left(1 - \frac{H}{V + A \cdot c' \cdot \cot\phi'}\right)^3 = \left(1 - \frac{H}{V}\right)^3 \tag{O.9}$$

Parameter	**Value**	**Unit**
H	35.6	kN
V	252	kN
i_q	2.66	[-]
i_γ	3.72	[-]

Calculation $p'_{max,drained}$

Parameter	Value	Unit
σ'_q	87.8	kN/m²
N_q	24.6	[-]
s_q	1.05	[-]
i_q	2.66	[-]
γ'	10	kN/m²
B	3.50	m
N_γ	21.9	[-]
s_γ	0.97	[-]
i_γ	3.72	[-]
$\mathbf{p'_{max,drained}}$	**7329**	**kN/m²**

O.1.3. Verification vertical stability

Parameter	Value	Unit
$\sigma_{k,max}$	80	kN/m²
$p'_{max,drained}$	7329	kN/m²
$\sigma_{k,max} < p'_{max}$		OK

O.2. Horizontal stability

The horizontal stability is in compliance if the sum of the horizontal forces exceed the friction force between the structure and the soil. The ratio between the total horizontal force and the vertical force relates to the friction between the bottom of the foundation and the soil. The friction for casted concrete on coarse sand should be around 0.55 (Manual Hydraulic Structures, 2023) or lower to prevent sliding. The resistance to sliding is assumed to be sufficient, observing if the following friction ratio does not exceed f.

$$\frac{\sum H}{\sum V} < f \tag{O.10}$$

The nett horizontal force on the foundation, i.e. the sum of the horizontal forces acting on the foundation, results in a force directed to the left hand side which is towards the opposite side of the main hydraulic loads. This is because of the passive soil pressure acting on the structure from the right hand side. This maximum pressure exceed the main loads which in practice can not occur. The soil mobilises as much as is required in order to be in equilibrium with the horizontal loads. This equilibrium occurs where the sum of the horizontal forces is equal to the friction force which is the sum of the vertical forces multiplied with the friction coefficient:

$$\sum H = f \cdot \sum V = 0.55 \cdot 161.7 = 88.94kN$$

The nett horizontal force on the foundation thus needs to be 88.94 kN. This means that in order for this horizontal force equilibrium to occur, the soil needs to mobilise as much that the nett soil pressure on the structure develops to have a resultant force which is 248.6 kN which provides the horizontal stability and is named H4 in Figure O.4. This force is important to take into account for the rotational stability check in the next section. The loads on the structure for this verification are summarised in Figure O.4.

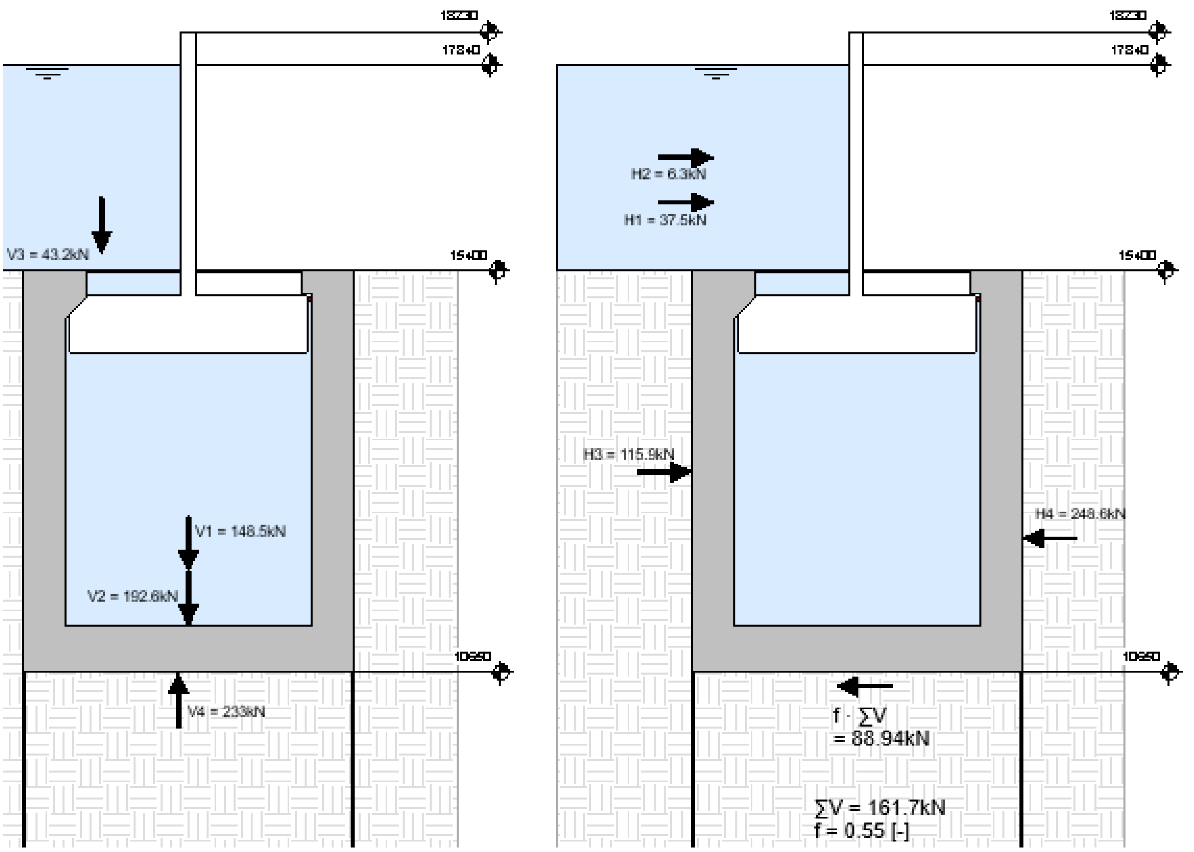


Figure O.4: Indication of all the loads acting on the structure for horizontal stability

O.3. Rotational stability

To ensure the rotational stability, the following requirement must be met:

$$e_R = \frac{\sum M}{\sum V} \leq \frac{b}{6} \tag{O.11}$$

where :	e_R	[m]	=	distance from middle of the structure to the intersection point of resultant force and the bottom line of structure
	$\sum V$	[kN]	=	the sum of vertical forces
	$\sum M$	[kNm]	=	the sum of acting moments around overturning point
	b	[m]	=	width of structure

The rotational stability verification is summarised in Figure O.5. Note horizontal force H4 from the horizontal stability verification which represents the nett horizontal soil pressure force on the structure, which means the sum of the passive and active soil pressure force.

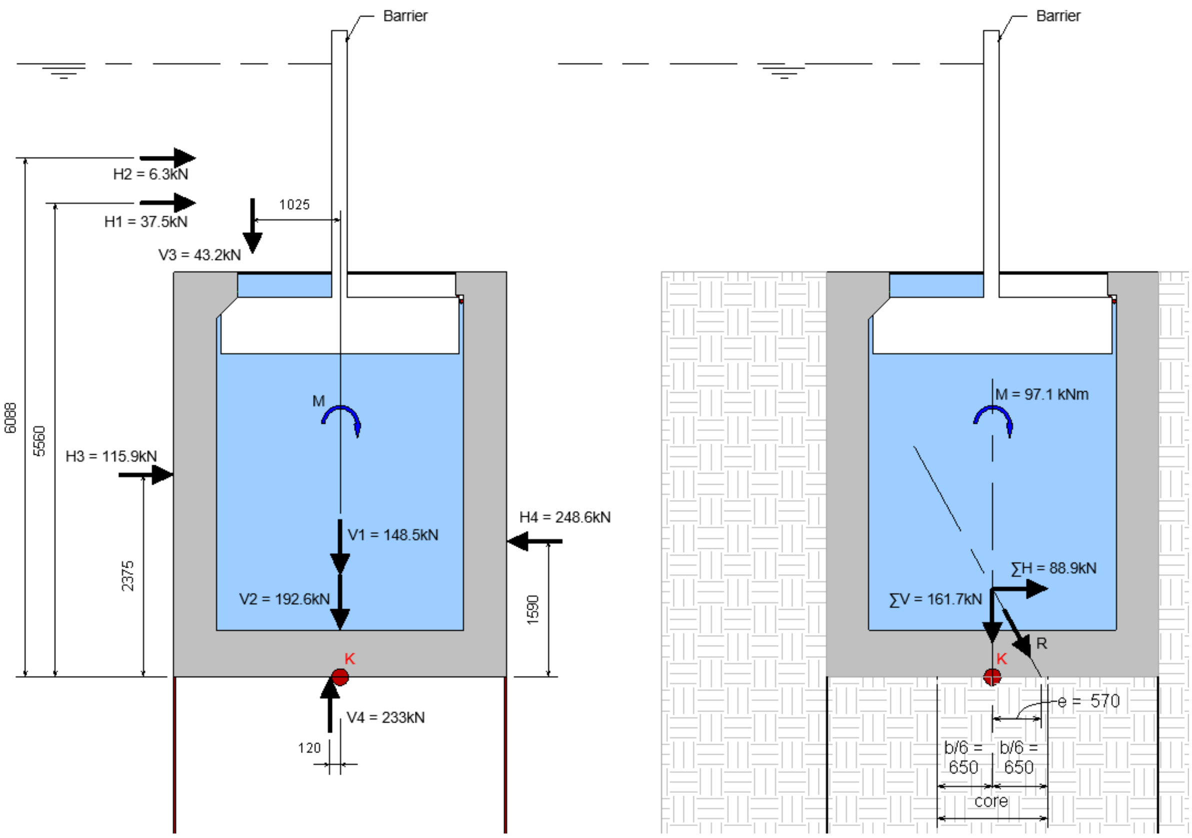


Figure O.5: Indication of all the loads acting on the structure for rotational stability

In Figure O.5 it can be observed that the rotational stability is in compliance which is numerically shown here:

$$e_R = \frac{\sum M}{\sum V} = \frac{97.1kNm}{161.7kN} = 0.57m \leq \frac{b}{6} = 0.65m$$

O.4. Uplift entire structure

The governing case is an empty floatation chamber and a water level at NAP + 13.65 m in the Meuse, which is approximately the water level where the floatation chamber starts to fill, leading to a saturated soil and a similar ground water level. The ground water pressure at the bottom of the foundation (NAP + 10.65 m) will be 30.0 kN/m². The width of the entire structure is 3.9 m. The upward force per unit span length is 117 kN/m as determined earlier. The design value of the weight of the structure in rest comprises of the gate weight and the weight of the concrete, which is in total 158 kN/m. The upward water force is 117 kN/m. Observing the verification below, uplift will not occur:

Parameter	**Value**	**Unit**
Factors		
k_{FI} (factor for upgrade to consequence class CC3)	1.1	[-]
γ_G (partial factor permanent load; favourable)	0.90	[-]
ξ (reduction factor for unfavourable self-weight)	0.89	[-]
Characteristic loads per unit span width		
Permanent loads (self-weight)		
$G_k = G_{gate} + G_c$	169	kN
Hydraulic loads		
$S_{d,1}$ (upward water force)	117	kN
Vertical design loads per unit span width		
$V_{1,d} = k_{FI} \cdot \gamma_G \cdot \xi \cdot G_k = 1.1 \cdot 0.90 \cdot 0.89 \cdot 169$	149	kN
$V_{2,d} = S_{d,1}$	117	kN
Verification uplift		
$\mathbf{V_{1,d} > V_{2,d}}$		**OK**

P

Gate design

In this Appendix, the complete gate design is elaborated including the design calculations. Firstly, the critical situation and failure mechanisms are outlined after which the geometry of the gate is described and the particular components as part of the gate assembly are designed on strength one by one.

P.1. Critical situation and failure mechanisms

The critical situation for the failure mechanisms instability and insufficient strength is the extreme event with high water of NAP + 17.84 m. The critical situation is schematised in Figure 5.12.

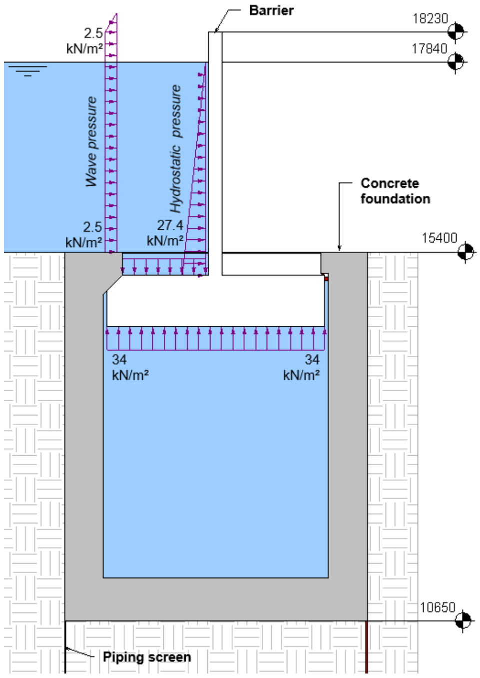


Figure P.1: Critical situation for the failure mechanisms instability and insufficient strength of the gate

Failure mechanism 1: instability of the gate in retaining position
The failure mechanism involving the instability of the gate in retaining position is illustrated in Figure 5.13. This phenomenon occurs if the gate is not locked in. This allows the gate to tilt and partially float out of the chamber because of the acting hydraulic loads from underneath the gate and against the retaining side of the gate.

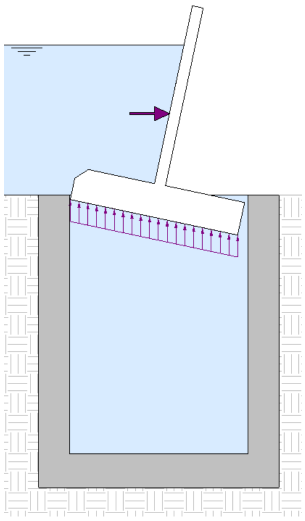


Figure P.2: Illustration of instability of the gate during the retaining function

Failure mechanism 2: insufficient strength of the gate components
The second failure mechanism of the gate is the insufficient strength of the components of the gate. This can be expressed by the criterion of Von Mises which should be valid for all points in all cross-sections:

$$\sqrt{\sigma_x^2 + 3 \cdot \tau^2} \leq f_y \tag{P.1}$$

where :	σ_x	[N/mm²]	=	stress in normal direction
	τ	[N/mm²]	=	shear stress
	f_y	[N/mm²]	=	yield stress

P.2. Geometry modelling

The main structural frame of the gate consists of the following components:

- Skin plate
- Horizontal stiffeners (girders)
- Vertical stiffeners (support columns)
- Floater box girder with compartment walls and rollers
- Top deck cover plate

All components are shown in Figure P.3, which shows a model of the gate. The various components are indicated that are required to provide sufficient strength and stiffness and an efficient distribution of loads. The main components of the gate are firstly the skin plate as this is the component that directly retains the water and secondly the floater box girder as this ensures the floatability of the gate. In order to distribute the acting loads efficiently from the skin plate to the concrete structure, the gate requires horizontal and vertical stiffeners, which act as girders and columns.

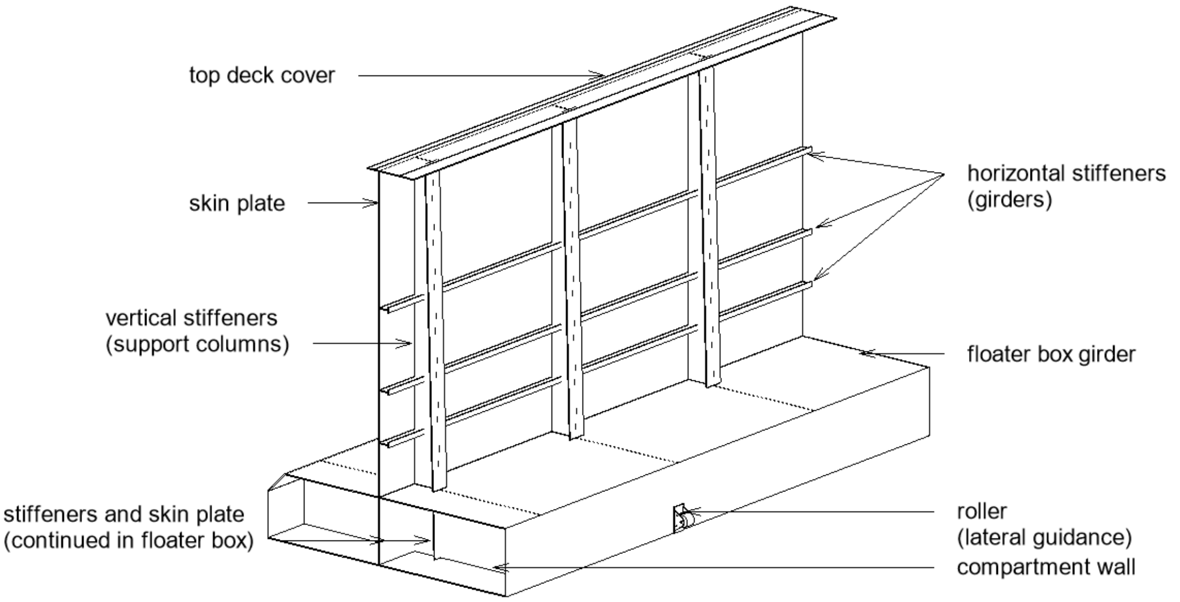
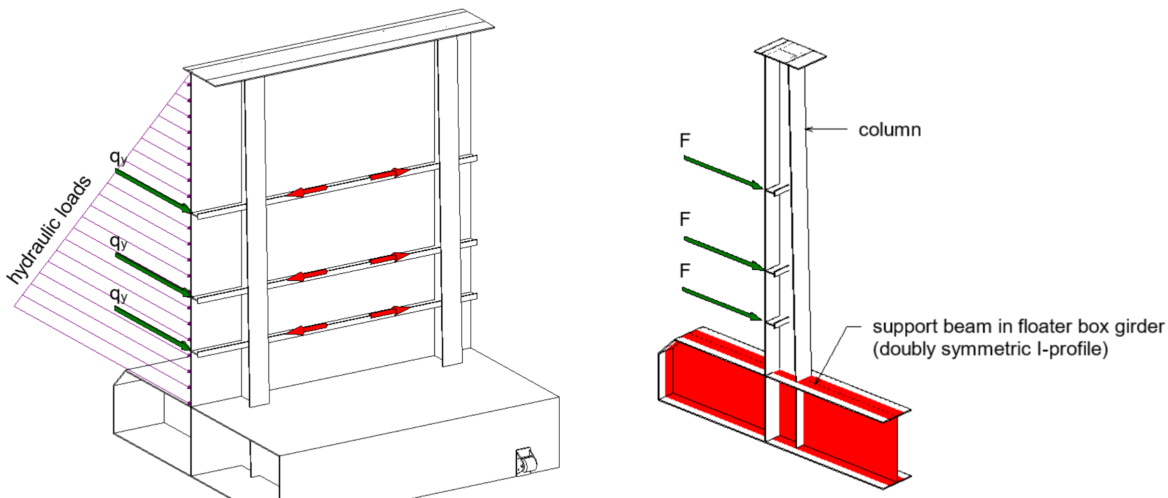


Figure P.3: 3D model of the gate

The girders increase the bending stiffness, divide the total hydraulic load and transfer it to the columns. This is illustrated in Figure P.4a. The columns subdivide the total gate span in subspans, which can be seen in Figure P.4a, and the columns transfer the loads to the compartment walls inside the floater box, which from a functional point of view are present to compartment the floater box girder to increase robustness of the system, but from a structural point of view are capable to also act as support beams for the columns to transfer the loads to. This is illustrated in Figure P.4b.



(a) Illustration of load transfer from the skin plate to the girders and in turn to the columns

(b) Indication of support beam in red with the compartment walls as the web and an effective width of the floater box as flanges

Figure P.4: Load directing model

In the next paragraphs, the components are dimensioned based on rule of thumbs from Erbisti.

Initial dimensions of skin plate

The skin plate is in fact the vertical wall that is directly in contact with the water when it is in the retaining position. The skin plate distributes the hydraulic loads to the rest of the components. Generally the minimal thickness of a skin plate is around 8 mm (Erbisti, 2014), that allows the plate to not be subjected to warping during welding of attached stiffeners. Warping occurs due to large temperature differences causing the plate to deform. Steel plates are often supplied with specific dimensions after which they are processed to the wishes of the client by a steel cutting company. Steel plates can be supplied with dimensions up to 12000x3000mm (Heus Staal, 2022), which is convenient for the assembly of the gate, since the retaining height of the barrier is 2.83 m. The height of the barrier directly determines the height of the skin plate. Thus per gate part of 40 m a minimum of four skin plates are needed and welded together to obtain the span of 40 m.

Verifying an initial plate thickness by a rule of thumb for the maximum plate bending stress

The book Design of Hydraulic Structures by Erbisti states a rule of thumb according to the Brazilian standard NBR-8883 for determining the plate bending stresses caused by water pressure which is based on the plate theory:

$$\sigma = \pm \frac{k}{100} \cdot p \cdot \frac{a^2}{t^2} \tag{P.2}$$

where :	σ	[kN/m²]	**=**	bending stress at support or midpoint of the plate
	k	[-]	=	factor depending on the ratio of the two support lengths and the location and direction of the bending stress
	p	[kN/m²]	=	water pressure at the center of a plate module
	a	[m]	=	minor support length
	t	[m]	=	plate thickness

Equation P.3 is solely used to get a first estimate of the skin plate thickness by checking the maximum plate bending stresses caused by the water pressure that is already known. The maximum of the factor k is $k = 75$, which is obtained from table 5.3 in the book Design of Hydraulic Structures. This value corresponds to σ_{xx} at the midpoint of the plate where the plate has a length over width ratio larger than 3, leading to the most unfavourable results. The minor support length will tentatively assumed to be 1 m, because this is yet to be determined with calculation of the required number of girders. For the thickness, 8 mm will be assumed for the aforementioned reason that it is generally minimally 8 mm thickness for steel plates. The maximum water pressure of $p = 24.4kN/m^2$ will be used. The result is given below:

Parameter	**Value**	**Unit**
k	75	-
p	24.4	kN/m²
a	1	m
t	$8 \cdot 10^{-3}$	m
$\sigma_{xx,mid}$	$286 \cdot 10^3$	kN/m²
$\sigma_{xx,mid}$	**286**	**N/mm²**

From the calculation follows, that a minimum thickness of $t = 8$ mm is applicable because the maximum stress values are in a range that is within a yield stress value of $f_y = 355$ N/mm² for steel grade class S355 which is common for structural steel elements. The exact stress values will be calculated in the structural analysis, but tentatively the thickness of $t = 8$ mm is considered a first good estimate.

Initial dimensions of girder

Attached to the skin plate are the girders in horizontal longitudinal direction. Girders are in fact stiff-eners, consisting of a web with a flange attached to it. The girders give the gate the actual bending moment resistance and shear force resistance in longitudinal direction. Furthermore, with girders, the total hydraulic load is subdivided into partial loads which makes it more efficient to resist and transfer the loads. Also, the cross-sectional gate width increases which means that the elastic section modulus also increases leading to more bending moment capacity. Girders also prevent the skin plate from extreme deflections and plate bending stresses, because the girders together with posts act as the supports of the plate dividing it in modules for which the plate bending stress can be calculated.

Number of girders

A first estimate for the number of girders is determined with the following empirical formula (Erbisti, 2014):

$$N = \frac{100 \cdot h}{t} \cdot \sqrt{\frac{H_m}{2 \cdot \sigma_{adm}}} \tag{P.3}$$

where :				
	N	[-]	**=**	number of horizontal girders rounded to integers
	H_m	[m]	=	water head in reference to the center point of the gate
	σ_{adm}	[N/mm²]	=	allowable bending stress
	h	[m]	=	gate height
	t	[mm]	=	plate thickness

Note that the units are not in accordance, because it is an empirical formula. Tentatively, steel grade class S355 is chosen for the girders which leads to a yield stress of 355 N/mm². The maximum allowable bending stress is the yield stress multiplied with a coefficient of table 5.1 in the book Design of Hydraulic Gates by Erbisti. These coefficients depend on the load case and the type of stress. For bending stress in an exceptional load case, the coefficient is 0.89. The water head is 2.44 m as determined in previous sections and the gate height is 2.83 m. The results of using equation P.3 gives a required number of girders of $N = 2.9$, which would mean 3 girders:

Parameter	**Value**	**Unit**
$H_m = 2.44 - \frac{2.83}{2}$	1.175	m
$\sigma_{adm} = 0.89 \cdot 355$	316	N/mm²
h	3.13	m
t	8	mm
N	**2.9**	**-**

Thus, there will be chosen for $N = 3$ girders. From the viewpoint of adaptibility, because the gates are adaptable to new hydraulic boundary conditions, this also means a future increase of the gate height. The future hydraulic boundary conditions might lead to the need for an additional girder.

Center-to-center spacing of girders

By choosing the locations of the centerlines wisely, the girders can be equally loaded, which leads to one specific cross-section to design for all girders. Locating the centerlines of each girder in the centroid of each area in the pressure diagram which is subdivided equivalently will lead to equally loaded girders. With the help of the following equation the distance from the top of the pressure to the centerline of the girder can be determined (Erbisti, 2014):

$$y_k = \frac{2 \cdot h}{3 \cdot \sqrt{n}} \cdot (k^{\frac{3}{2}} - (k-1)^{\frac{3}{2}}) \tag{P.4}$$

where : y_k	[m]	=	distance of centerline of girder with respect to the top of the pressure diagram
n	[-]	=	number of girders
h	[m]	=	height of gate segment loaded by the water pressure
k	[-]	=	girder number from top to bottom {1,2,3,...,n}

Parameter	**Value**	**Unit**
Distance centerline of girder with respect to the still water level		
y_1	0.810	m
y_2	1.810	m
y_3	2.460	m
Distance of centerline of girder with respect to the top of the gate		
y_1	1.200	m
y_2	2.200	m
y_3	2.850	m

Dimensions of girder

The following step is to determine the dimensions of the girder which are:

1. Web thickness (t_w):
   The minimum recommended web thickness is 8 mm (Erbisti, 2014). Thus, as a first estimate a web thickness of $t_w = 8$ mm is chosen.

2. Web depth (h_w):
   The web depth is preliminary determined to be $\frac{1}{12}$ of the girder support length. This is obtained from table 5.5 of the book Design of Hydraulic Structures which give empirical relations for the web depth based on the water head. Tentatively assuming a girder support length of 2 m, would to a web depth of $h_w = 167$ mm. At first sight, this web depth seems too large. As a first estimate will be chosen for the half of this value, which is $h_w = 84$ mm

3. Flange thickness (t_f):
   The flange thickness is generally equal or larger than the thickness of the web. Thus, a flange thickness of $t_f = 8$ mm is chosen.

4. Flange width (b_f):
   For the flange width $\frac{1}{5}$ of the web depth can be taken as first good estimate for preliminary calculations (Erbisti, 2014). This results in a flange width of $b_f = 20$ mm. However, considering the ratio between the girder height and width, it does not agree with the same ratio for an equivalent IPE profile. It might be useful to more or less follow standard profile dimensions. For this reason, a flange width of $b_f = 50$ mm will be chosen. This dimension will still leave enough space for access regarding welding of the elements.

Choosing dimensions for vertical column stiffeners
The girders as described in the previous paragraph transfer the loads to the vertical stiffeners. The vertical stiffeners are T-shaped profiles that are also welded to the skin plate, making a column with an H- or I-shaped profile and the girders. The stiffeners act as the column supports of the girders and divide the total gate span into subspans to reduce the internal forces in the girders. They can be either schematised as fixed or simple supports, depending on the design and execution. The posts will have a height that is equal to the retaining height, which is 3.13 m and they transfer the loads further to the support beam in the floater box.

There is preliminary chosen to have spans of 2 m as an initial starting point. The thickness of the posts are set equal to the girder which is $t = 8$ mm. The cross-sections of the columns will have a variable height over the span height, because the highest loads will occur at the bottom of the column. As a starting point, the gate width will be $w = 0.2$ m at the top and 0.3 m at the bottom.

Dimensions of floater box girder with compartment walls and rollers
The floater box is the bottom part of the gate which is not water retaining and is initially purely functional because it provides the required buoyancy. However, the floater box girder is compartmentalised which means that the columns are connected to a relative high support beam with an effective width. These support beam have conveniently a large bending capacity and transfer the loads in turn to the concrete. The thickness of all parts of the floater box girder is initially set to t = 8 mm.

Summary of dimensions of cross-section
In Figure P.5 the preliminary dimensioning of the previous sections is summarised.

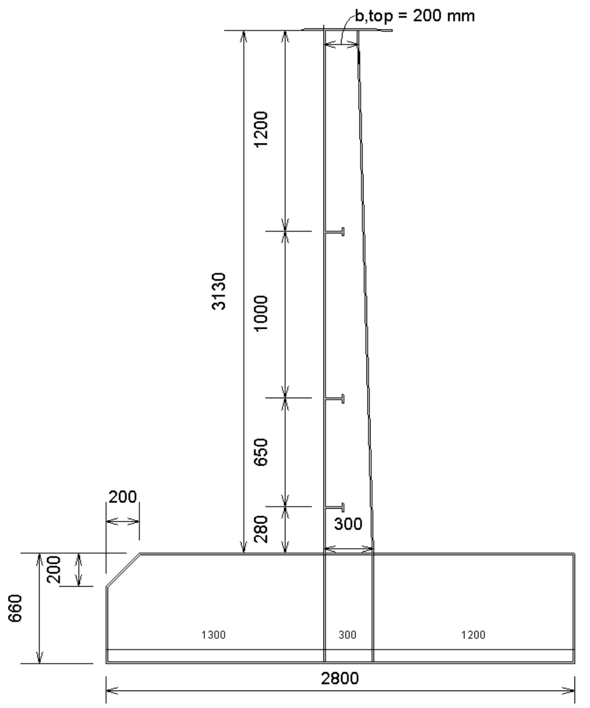


Figure P.5: Initial dimensions of the cross-section of the assembled gate

The structural design of the gate will continue per main (governing) component. Each following subsection will elaborate on the design of each main component in the following order:

- Strength of girder
- Strength of column
- Strength of support beam

- Plate bending stress (governing module)

The elaboration of the structural design of each component involves the specification of the failure mechanisms with associated limit state functions following the Eurocode, then determining the geometric and cross-sectional properties, followed by translating the total load on the gate to loads per element. After this the structural analysis will be performed in which each component is verified on its structural integrity related to for example the maximum stress values, buckling and deformation.

P.3. Strength of girder

Failure mechanisms and limit states

In the high water load situation, the hydraulic pressure load is transferred via the skin plate to the girder in the form of a partial line load working in the y-direction and leading to bending stresses around the z-axis. Furthermore, only self-weight comes into play, acting in the z-direction and leading to bending stresses around the y-axis. As mentioned before in the section about dimensioning of the girders, all girders are equally loaded by the positioning in vertical direction relative to each other. The failure mechanisms for this structural component are therefore:

- Exceedance of bending moment resistance in governing cross-sections (ULS)
- Exceedance of shear resistance in y-direction in governing cross-sections (ULS)
- Exceedance of shear resistance in z-direction in governing cross-sections (ULS)
- Extreme deflections midspan (SLS)

The girder is not susceptible to lateral torsional buckling according to clause 6.3.2.1 (2) of NEN-EN 1993-1-1 in which is stated that girders with sufficiently supported compression-loaded flanges are not susceptible to lateral torsional buckling. The compression loaded flange in this case is an effective width of the skin plate that behaves as the compression flange, but is therefore fully supported along the entire span.

For these failure mechanisms the following limit state functions are determined for the verification:

NEN-EN 1993-1-1 6.2.1(7):

$$Z_1 = \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} - 1 \qquad Z_1 \leq 0 \qquad \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

NEN-EN 1993-1-1 6.2.6(1):

$$Z_2 = \frac{V_{y,Ed}}{V_{y,c,Rd}} - 1 \qquad Z_2 \leq 0 \qquad \frac{V_{y,Ed}}{V_{y,c,Rd}} \leq 1$$

NEN-EN 1993-1-1 6.2.6(1):

$$Z_3 = \frac{V_{z,Ed}}{V_{z,c,Rd}} - 1 \qquad Z_3 \leq 0 \qquad \frac{V_{z,Ed}}{V_{z,c,Rd}} \leq 1$$

Bouwen met Staal (2007):

$$Z_4 = \frac{w_{mid}}{\frac{1}{250} \cdot L} - 1 \qquad Z_4 \leq 0 \qquad \frac{w_{mid}}{\frac{1}{250} \cdot L} \leq 1$$

The verification to bending combined with shear will be considered in the verification of shear resistance.

Geometry of cross-section

An effective width of the skin plate acts as the compression or tension flange of the girder, depending on the moment distribution. The effective width fluctuates along the span with the bending moment diagram. As a compression flange, a larger effective width can be taken into account whereas as a tension flange a smaller effective width can be taken into account. The extremes are in Figure P.6 denoted with $\gamma_I \cdot B$ at midspan of the girder for a compression flange width and $\gamma_{II} \cdot B$ at the supports of the girder for tension flange width.

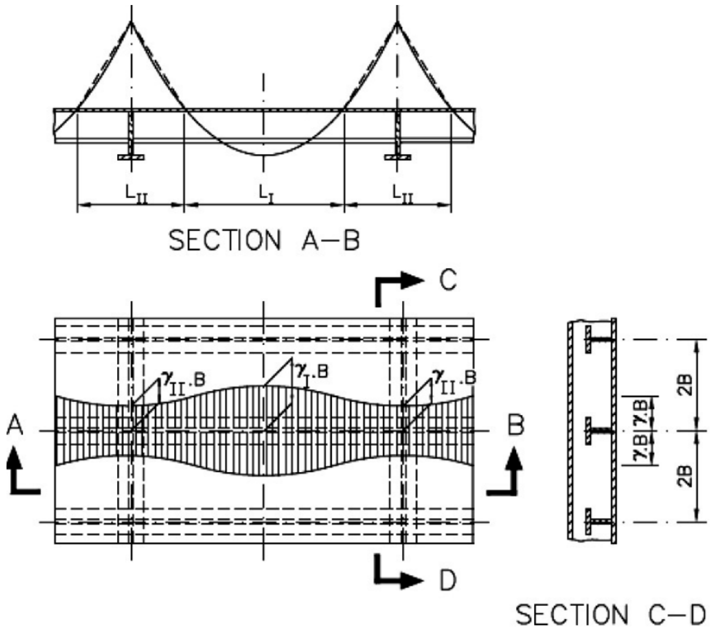


Figure P.6: Effective width of skin plate (Erbisti, 2014)

The associated reduction factors γ_I or γ_{II} can be determined with Figure P.7 in which the factors are plotted against the ratio of the effective length L of the girder in which the skin plate act as either a compression or tension flange over the width B which is half the centre-to-centre spacing between two girders.

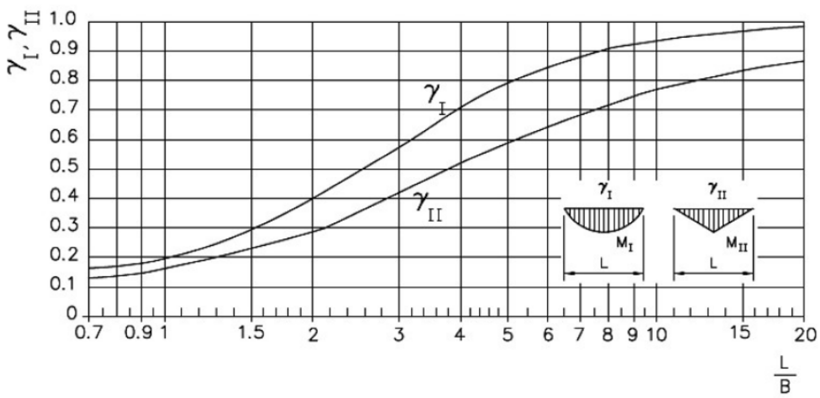
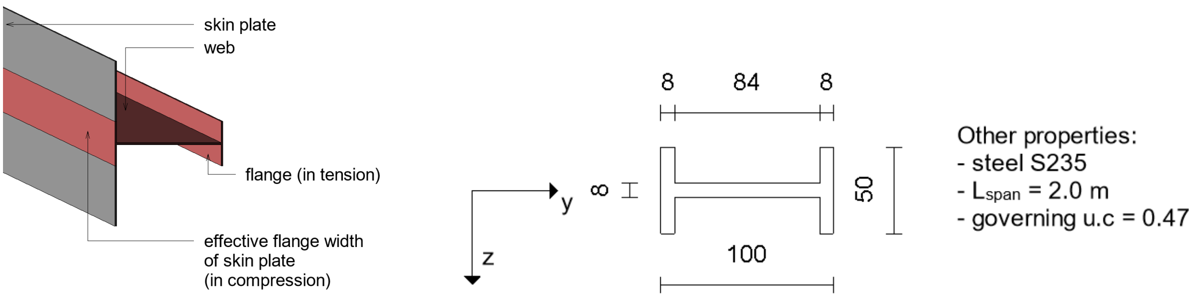


Figure P.7: Reduction factor for co-acting width of skin plate (Erbisti, 2014)

However, conservatively will be assumed that the effective width of the skin plate acting as a flange of the girder is equal to the other flange width as determined when dimensioning the girder, which was $b_f = 50$ mm. The effective width is in practice thus larger but this will not be considered here. The cross-section will consequently be considered as a doubly-symmetric flanged built-up beam with a continuous span length of $L = 40.0$ m supported by columns every 2 m. The total height is $h = 100$ mm and the thickness of the web and flanges are $t = 8$ mm. The steel grade is chosen to be S235. The geometry of the cross-section of the girder is shown in Figure P.8. The girder is shown in red which actively will resist the loads. In grey the remaining part of the skin plate is shown which will conservatively will not be taken into account for the girder design.



(a) Indication of girder (b) Cross-section of girder

Figure P.8: Model of girder

Classification of the cross-section
The role of the classification is to identify the extent to which the resistance and rotation capacity of the cross-section are limited by local buckling of its parts. The classification of the cross-section is done according to NEN-EN-1993-1-1 Table 5.2 which is shown in Figure P.9.

Width to thickness limits for cross-section classification according to EN1993-1-1 Table 5.2

Class	Web		Outstand Flanges
	Web in pure compression	Web in pure bending	Flanges in pure compression due to axial force or bending moment
Class 1	$c / t \leq 33\varepsilon$	$c / t \leq 72\varepsilon$	$c / t \leq 9\varepsilon$
Class 2	$c / t \leq 38\varepsilon$	$c / t \leq 83\varepsilon$	$c / t \leq 10\varepsilon$
Class 3	$c / t \leq 42\varepsilon$	$c / t \leq 124\varepsilon$	$c / t \leq 14\varepsilon$

Figure P.9: Width to thickness limits for cross-section classification according to EN1993-1-1 Table 5.2 (Eurocode, 2023)

where ϵ is a factor that takes into account the yield stress which is determined by:

$$\epsilon = (\frac{235}{f_y})^{0.5}$$

Since the steel grade class is S235, $\epsilon = 1.0$.

Flange classification
For the compression flange holds:

$$\frac{c}{t} = \frac{\frac{b-t_w}{2}}{t_f} = \frac{\frac{50-8}{2}}{8} = 1.625$$

$$\frac{c}{t} = 1.625 \leq 9 \cdot \epsilon = 9 \cdot 1.0 = 9.0$$

The flange is classified as class 1. In Figure P.10 the parameters that are used for the classification are indicated.

Web classification
For the web holds:

$$\frac{c}{t} = \frac{h_w}{t_w} = \frac{84}{8} = 10.5$$

$$\frac{c}{t} = 10.5 \leq 72 \cdot \epsilon = 72 \cdot 1.0 = 72.0$$

The web is classified as class 1. In Figure P.10 the parameters that are used for the classification are indicated.

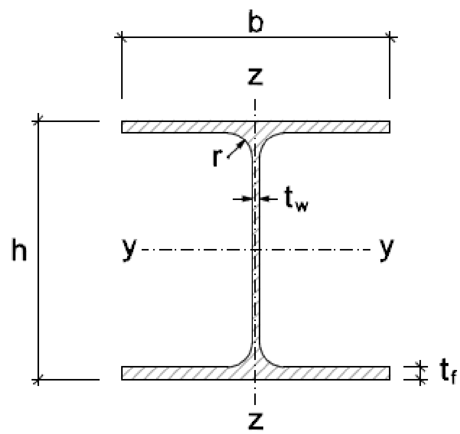


Figure P.10: Notation for flanged profiles according to EN1993-1-1 (Eurocode, 2023)

Classification of total cross-section
The class of the total cross-section corresponds to the most adverse of the flange class (class 1) and the web class (class 1). Therefore the cross-section is classified as Class 1. This means that plastic bending moment resistance develops and plastic hinge develops with rotation capacity adequate for plastic analysis, so plastic calculation may be performed.

Determination of cross-sectional properties
The associated cross-sectional properties for a doubly-symmetric flanged profile are given by the following relations. In Figure P.10 the parameters are indicated that are used in these relations.

$$h = h_w + 2 \cdot t_f \tag{P.5}$$

$$I_y = \frac{b_f \cdot h^3 - ((b_f - t_w) \cdot h_w^3)}{12} \tag{P.6}$$

$$I_z = \frac{t_f \cdot b_f^3}{6} + \frac{h_w \cdot t_w^3}{12} \tag{P.7}$$

$$W_y = \frac{I_y}{\frac{b}{2}} \tag{P.8}$$

$$W_z = \frac{I_z}{\frac{h}{2}} \tag{P.9}$$

$$A = 2 \cdot b_f \cdot t_f + h_w \cdot t_w \tag{P.10}$$

$$A_{v,z} = A - h_w \cdot t_w \tag{P.11}$$

$$A_{v,y} = \eta \cdot h_w \cdot t_w \tag{P.12}$$

The results of determining the cross-sectional properties are:

Parameter	Value	Unit
I_y	$0.172 \cdot 10^6$	mm^4
I_z	$2.181 \cdot 10^6$	mm^4
$W_{pl,y}$	$11.66 \cdot 10^3$	mm^3
$W_{pl,z}$	$53.12 \cdot 10^3$	mm^3
A	$1.53 \cdot 10^3$	mm^2
$A_{v,z}$	$0.806 \cdot 10^3$	mm^2
$A_{v,y}$	$0.855 \cdot 10^3$	mm^2

Design loads on girder
In Figure P.11 the nett effective hydraulic loads per unit meter span are indicated and denoted with H1 and H2, which are the hydrostatic pressure load of the still water level and respectively the static wave pressure load.

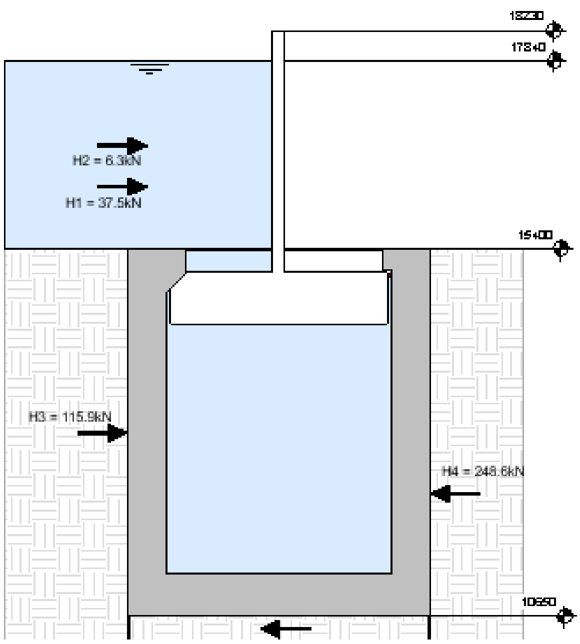


Figure P.11: Horizontal loads

Loads in y-direction
The load q_y on the girder is the total horizontal hydraulic load divided by the number of girders $N = 3$ which results in:

$$q_{y,Ed} = \frac{H}{N} = \frac{H_1 + H_2}{N} = \frac{37.5 + 6.3}{3} = \frac{43.8}{3} = 14.6kN/m$$

Note that for the hydraulic loads no safety factors are included, since uncertainty is already discounted using the probabilistic approach.

Loads in z-direction
The characteristic load q_z on the girder is the self-weight of the girder which is:

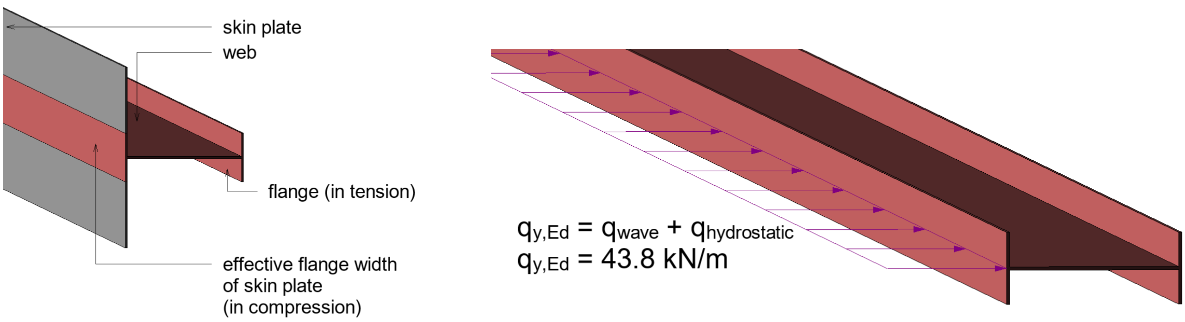
$$q_{z,k} = \rho_s \cdot A = 78.50 \cdot 1.53 \cdot 10^3 \cdot 10^{-6} = 0.12kN/m$$

The design load q_z on the girder is then based on NEN-EN-1990-1-1 (6.10b) :

$$q_{z,d} = k_{FI} \cdot \gamma_G \cdot \xi \cdot q_{z,k} = 1.1 \cdot 1.35 \cdot 0.89 \cdot 0.12 = 0.16kN/m$$

The magnitude of the self-weight is too insignificant and therefore the bending moment and shear verification in z-direction will be omitted.

In Figure P.12 the design loads on the girder are shown.



(a) Indication of girder

(b) Loads on gate

Figure P.12: Model of girder

The girders are schematised as continuous beams intermediately supported by each vertical column stiffener every 2 m. This is schematised in Figure P.13.

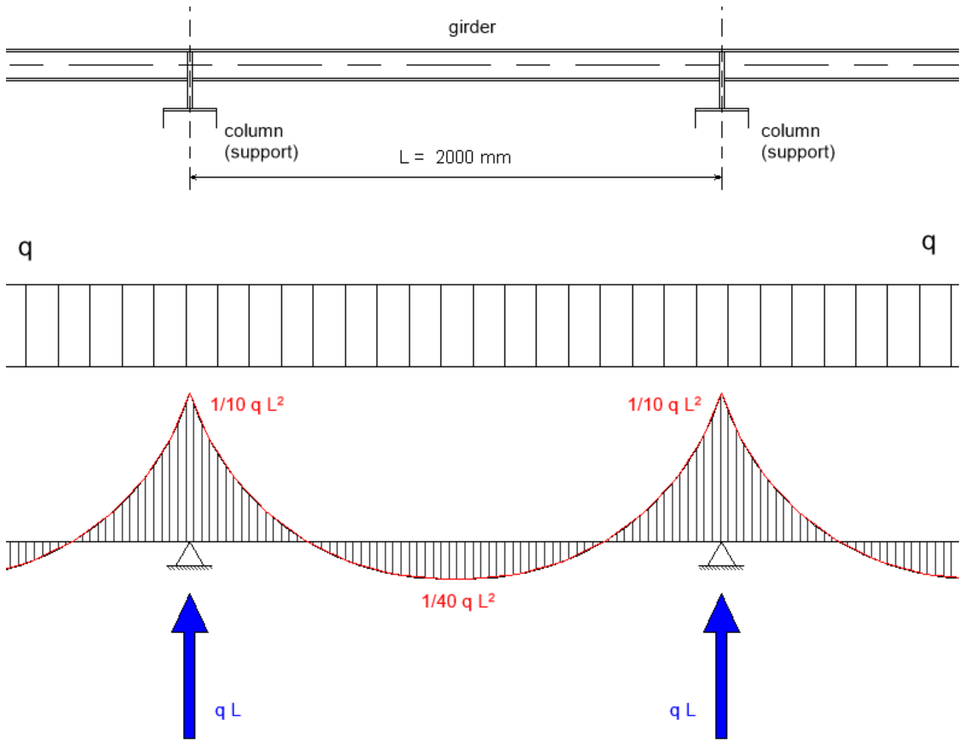


Figure P.13: Structural mechanics scheme of girder

Bending moment verification
The governing cross-section is at the supports as can be seen in Figure P.13. For this cross-section the unity check will be performed. The bending moment verification is shown in the following table.

Parameter	**Value**	**Unit**
Load effect		
$M_{z,Ed}$	5.84	kNm
Bending moment resistance		
$W_{pl,z}$ f_y γ_{M0} $M_{z,Rd} = \frac{W_{pl,z} \cdot f_y}{\gamma_{M0}}$	$53.12 \cdot 10^3$ 235 1.0 12.48	mm^3 N/mm^2 - kNm
Unity check		
$U.C. = \frac{M_{z,Ed}}{M_{z,Rd}}$	0.47	-
Verification NEN-EN-1993-1-1 6.2.5		
U.C. $<$ 1.0		**OK**

Shear resistance verification

The maximum shear force is at the intermediate support columns which is the governing cross-section is at the supports of the girder. For this cross-section the unity check will be performed. The shear force verification is shown in the following table.

Parameter	**Value**	**Unit**
Load effect		
$V_{y,Ed}$	14.6	kN
Bending moment resistance		
$A_{v,y}$ f_y γ_{M0} $V_{y,Rd} = \frac{A_{v,y} \cdot \frac{f_y}{\sqrt{3}}}{\gamma_{M0}}$	$0.806 \cdot 10^3$ 235 1.0 109.4	mm^2 N/mm^2 - kN
Unity checks		
$U.C. = \frac{V_{y,Ed}}{V_{y,Rd}}$	0.13	-
Verification NEN-EN-1993-1-1 6.2.6		
U.C. $< \mathbf{1.0}$		**OK**

Combination bending and shear

According to EN1993-1-1 6.2.8(3) the bending resistance of the cross-section is reduced when the applied shear force V_{Ed} is larger than one-half of the corresponding plastic shear resistance V_{Rd}.

In this case, it is:

Shear force along axis y-y: $\frac{V_{y,Ed}}{V_{y,Rd}} = \frac{14.6kN}{109.4kN} = 0.13 \leq 0.50$

The applied shear force $V_{y,Ed}$ is less than 50% of the corresponding plastic shear resistance. Therefore the effect of shear forces on the bending moment resistance may be ignored according to EN1993-1-1 6.2.8(3).

Shear buckling

The shear buckling resistance of the web is verified in accordance with NEN-EN-1993-1-1 6.2.6(6):

$$\frac{h_w}{t_w} \leq \frac{72 \cdot \epsilon}{\eta}$$

$$\frac{h_w}{t_w} = \frac{84}{8} = 10.5 \leq \frac{72 \cdot 1.0}{1.2} = 60$$

Shear buckling resistance is sufficient.

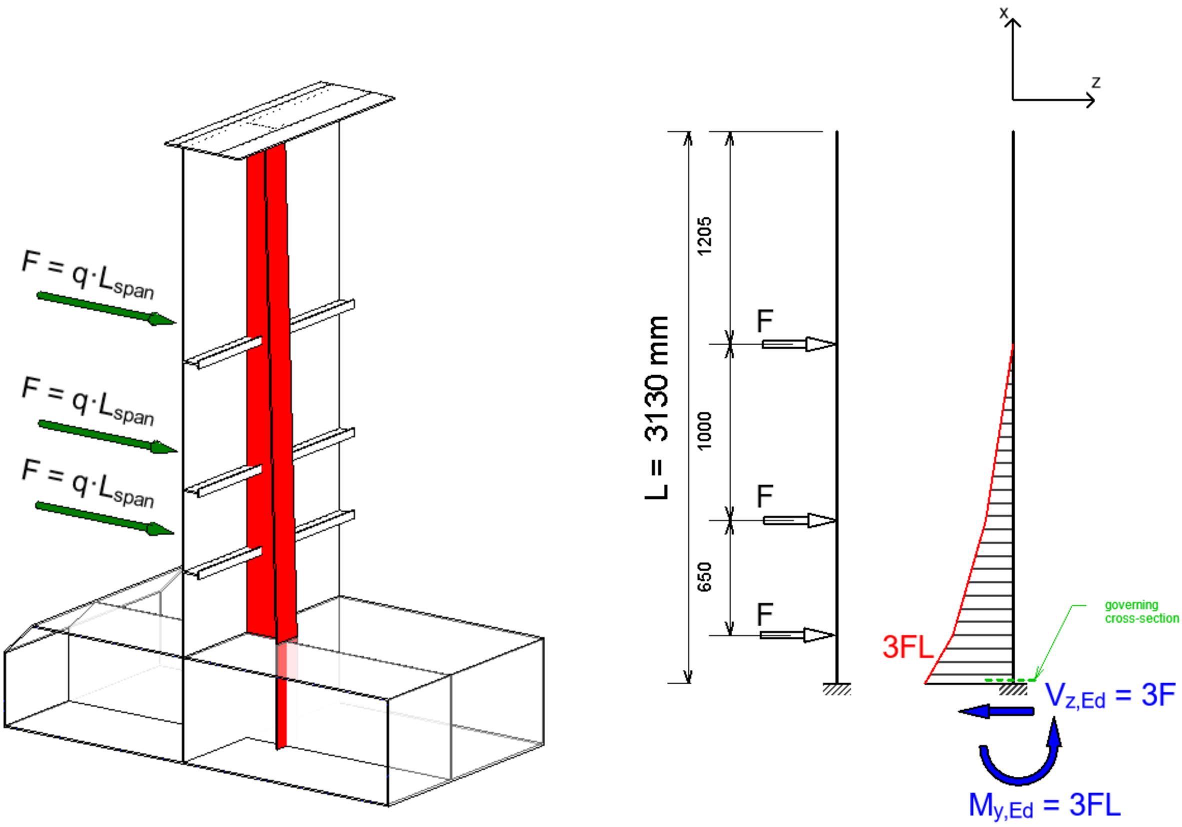
Deflection verification

The deflections will solely be checked for the plate modules, as these are the most governing with respect to deflection.

P.4. Strength of column

Failure mechanisms and limit states

The loads from the girder in the previous Section P.3 result in support reactions which are the direct loads on the columns. This means that the columns are subjected to three point loads in the y-direction leading to bending stresses around the z-axis. Furthermore, only self-weight comes into play, acting in the z-direction and leading to bending stresses around the y-axis. The model with the acting loads is illustrated in Figure P.14.



(a) Indication of column (b) Loads on column

Figure P.14: Model of column

The failure mechanism is the exceedance of the yield stress in all the cross-sections over the height the column by the general plane stress. This theoretical stress is described in the NEN-EN-1993-1-1 6.2.1 (5) with the Von-Mises criterion. The loads acting on the column result in normal stress in only one direction, which simplifies the criterion to:

$$\sqrt{\sigma_x^2 + 3 \cdot \tau^2} \leq f_y \tag{P.13}$$

where :	σ_x	[N/mm²]	=	stress in normal direction
	τ	[N/mm²]	=	shear stress
	f_y	[N/mm²]	=	yield stress

The column is not susceptible to lateral torsional buckling according to clause 6.3.2.1 (2) of NEN-EN 1993-1-1 in which is stated that girders with sufficiently supported compression-loaded flanges are not susceptible to lateral torsional buckling. The compression loaded flange in this case is an effective width of the skin plate that behaves as the compression flange, but is therefore fully supported along the entire span.

Geometry of cross-section

The geometry of the cross-sections of the column are shown in Figure P.15, where the cross-section at the top is shown and the cross-section at the bottom is shown, because, as was already mentioned, the cross-section of the column is variable over the height. The flange on the left hand side is again an effective width of the skin plate acting as a flange for the column. The dimensions of the cross-sections are initially assumed.

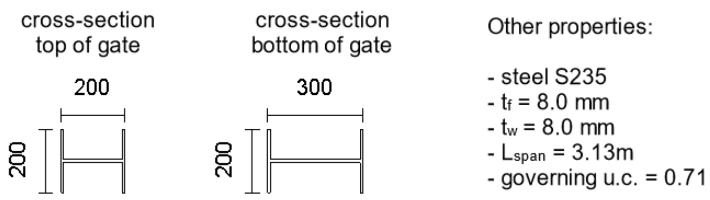


Figure P.15: Cross-sections of column

Classification of the cross-section

The role of the classification is to identify the extent to which the resistance and rotation capacity of the cross-section are limited by local buckling of its parts. The classification of the cross-section is done according to NEN-EN-1993-1-1 Table 5.2 which was already shown in Figure P.9 with the classification of the cross-section of the girder. The steel grade class is S235, meaning $\epsilon = 1.0$.

Flange classification

For the compression flange holds:

$$\frac{c}{t} = \frac{\frac{b-t_w}{2}}{t_f} = \frac{\frac{200-8}{2}}{8} = 12$$

$$\frac{c}{t} = 12 \leq 14 \cdot \epsilon = 14 \cdot 1.0 = 14.0$$

The flange is classified as class 3.

Web classification

For the web holds:

$$\frac{c}{t} = \frac{h_w}{t_w} = \frac{284}{8} = 35.5$$

$$\frac{c}{t} = 35.5 \leq 72 \cdot \epsilon = 72 \cdot 1.0 = 72.0$$

The web is classified as class 1.

Classification of total cross-section

The class of the total cross-section corresponds to the most adverse of the flange class (class 3) and the web class (class 1). Therefore the cross-section is classified as Class 3. This means that elastic calculation may be performed.

Determination of cross-sectional properties

The associated cross-sectional properties for a doubly-symmetric flanged profile are given by the aforementioned Equations P.5 up to and including P.12.

Only the properties along the major axis are relevant. The results of determining the cross-sectional properties are:

Parameter	Value	Unit
I_y	$84.58 \cdot 10^6$	mm^4
$W_{el,y}$	$563.9 \cdot 10^3$	mm^3
A	$5.527 \cdot 10^3$	mm^2
$A_{v,z}$	$2.726 \cdot 10^3$	mm^2

Design loads on column
In Figure P.14 the design loads on the column were already shown.

The loads F on the column are the support reactions from the girders:

$$F = q_{y,Ed} \cdot L_{span} = 14.6 \cdot 2.0 = 29.2kN/m$$

The self-weight of the column is taken into account in the structural calculations in MatrixFrame but are not shown in Figure P.14.

The verification of the Von-Mises criterion
The columns are verified with the help of the software Matrixframe. The cross-section varies over the height of the columns and with Matrixframe it is easier to find the governing cross-section and the maximum stress point. For the governing cross-section, which is at the support of the column, the maximum Von-Mises stress is calculated which is lower than the yield stress:

$$\sqrt{\sigma_x^2 + 3 \cdot \tau^2} = 166.8N/mm^2 \leq f_y = 235N/mm^2 \qquad \text{(P.14)}$$

The associated unity check for this verification is consequently, U.C. = 0.71 and is therefore within the acceptable limit. A hand calculation for this verification is also done to validate the result, but is omitted in this report. The results of the calculation in MatrixFrame are shown in Figure P.16.

Deflections
The deflections of the columns are negligible and are therefore omitted in this report.

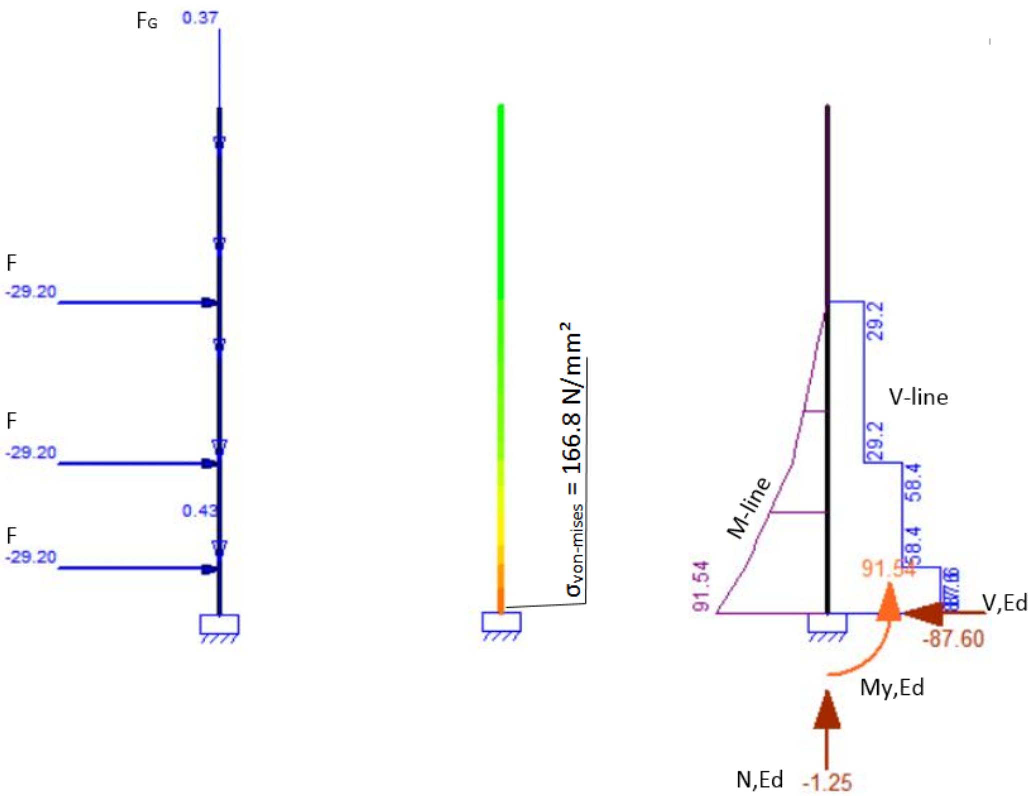


Figure P.16: MatrixFrame model schemes: left figure is the structural mechanics scheme; centre figure is a Von-Mises stress diagram with a colour scale where red means a high stress and green means a low stress value with respect to the yield stress; right figure is a diagram with the moment and shear force distribution and the support reactions.

P.5. Strength of support beam

Failure mechanisms and limit states

The support reactions from the column in the previous Section P.4 and the top and bottom water pressure on the floater box are directed the support beams. The support beam and its connection with the column is shown in Figure P.17.

The failure mechanism is the exceedance of the yield stress in all the cross-sections over the span of the support beam by the general plane stress. This theoretical stress is described in the NEN-EN-1993-1-1 6.2.1 (5) with the Von-Mises criterion. The loads acting on the column result in normal stress in only one direction, which simplifies the criterion to:

$$\sqrt{\sigma_x^2 + 3 \cdot \tau^2} \leq f_y \tag{P.15}$$

where :	σ_x	[N/mm²]	=	stress in normal direction
	τ	[N/mm²]	=	shear stress
	f_y	[N/mm²]	=	yield stress

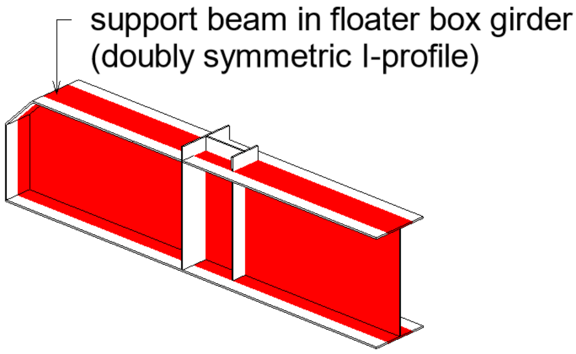


Figure P.17: Indication of support beam

The beam is not susceptible to lateral torsional buckling according to clause 6.3.2.1 (2) of NEN-EN 1993-1-1 in which is stated that girders with sufficiently supported compression-loaded flanges are not susceptible to lateral torsional buckling. The compression loaded flange in this case is an effective width of the continuous plates of the box girder that behaves as the compression flange, but is therefore fully supported along the entire span.

Geometry of cross-section

The geometry of the cross-sections of the beams are shown in Figure P.18. The height follows from the functional design. As an effective width 200 mm is chosen and the thickness is set equal to the rest of the steel components which is t = 8 mm.

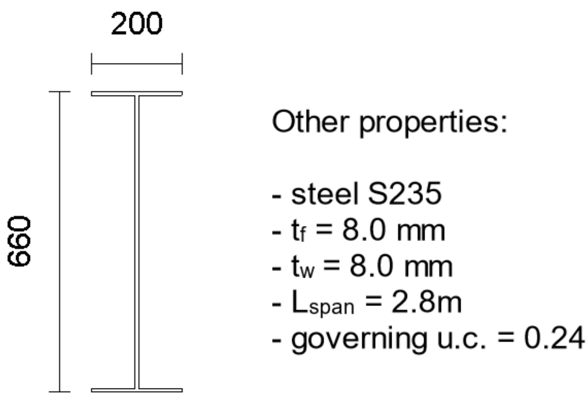


Figure P.18: Cross-section of support beam

Classification of the cross-section

The role of the classification is to identify the extent to which the resistance and rotation capacity of the cross-section are limited by local buckling of its parts. The classification of the cross-section is done according to NEN-EN-1993-1-1 Table 5.2 which was already shown in Figure P.9 with the classification of the cross-section of the girder. The steel grade class is S235, meaning $\epsilon = 1.0$.

Flange classification

For the compression flange holds:

$$\frac{c}{t} = \frac{\frac{b-t_w}{2}}{t_f} = \frac{\frac{200-8}{2}}{8} = 12$$

$$\frac{c}{t} = 12 \leq 14 \cdot \epsilon = 14 \cdot 1.0 = 14.0$$

The flange is classified as class 3.

Web classification
For the web holds:

$$\frac{c}{t} = \frac{h_w}{t_w} = \frac{660 - 8 - 8}{8} = 80.5$$

$$\frac{c}{t} = 80.5 \leq 83 \cdot \epsilon = 83 \cdot 1.0 = 83.0$$

The web is classified as class 2.

Classification of total cross-section
The class of the total cross-section corresponds to the most adverse of the flange class (class 3) and the web class (class 2). Therefore the cross-section is classified as Class 3. This means that elastic calculation may be performed.

Determination of cross-sectional properties
The associated cross-sectional properties for a doubly-symmetric flanged profile are given by the aforementioned Equations P.5 up to and including P.12.

Only the properties along the major axis are relevant. The results of determining the cross-sectional properties are:

Parameter	**Value**	**Unit**
I_y	$518.2 \cdot 10^6$	mm^4
$W_{el,y}$	$1570 \cdot 10^3$	mm^3
A	$8.353 \cdot 10^3$	mm^2
$A_{v,z}$	$6.182 \cdot 10^3$	mm^2

Design loads on beam
In Figure P.19 the design loads on the support beam are shown. The self-weight of the entire steel barrier is taken into account and is denoted in Figure P.19 as F_G. The moment and horizontal force are resulting support reacting from the columns transferred to the beam. Lastly, the uniformly distributed loads, denoted with $q_{z,Ed}$ are the water pressure loads that directly act on the floater towards the support beam.

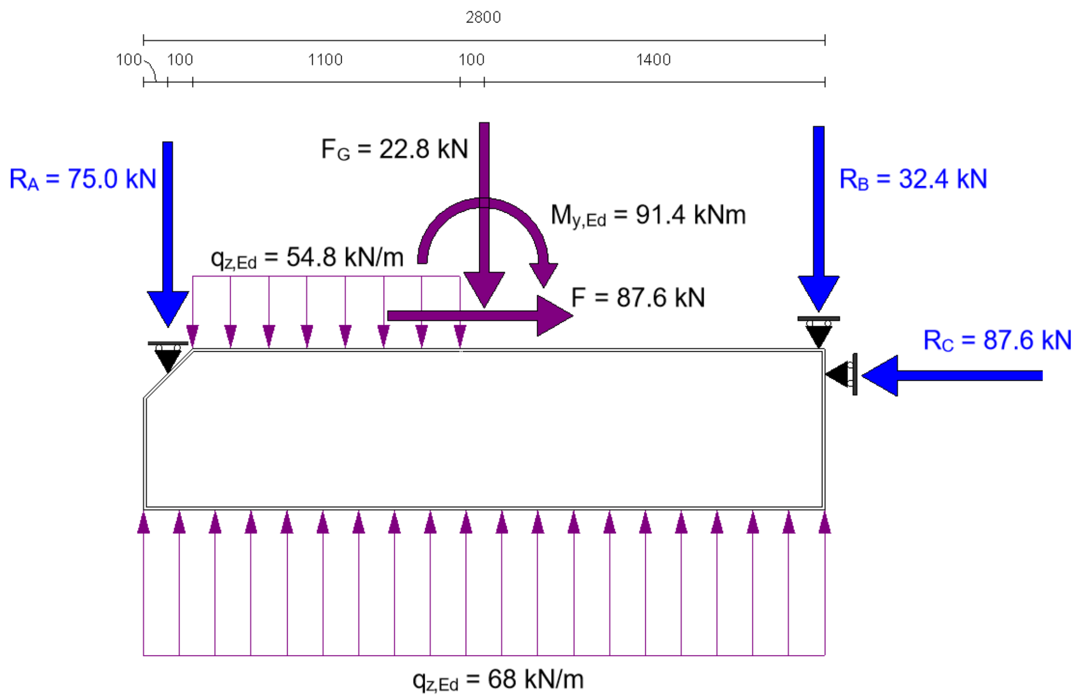


Figure P.19: Structural mechanics scheme of loads on support beam

The verification of the Von-Mises criterion

The columns are verified with the help of the software Matrixframe. The cross-section varies over the height of the columns and with Matrixframe it is easier to find the governing cross-section and the maximum stress point. For the governing cross-section, which is at the support of the column, the following Von Mises stress is calculated which is lower than the yield tress:

$$\sqrt{\sigma_x^2 + 3 \cdot \tau^2} = 56 N/mm^2 \leq f_y = 235 N/mm^2 \tag{P.16}$$

The associated unity check for this verification is consequently, U.C. = 0.24 and is therefore within the acceptable limit. A hand calculation for this verification is also done to validate the result, but is omitted in this report. The results of the calculation in MatrixFrame are shown in Figures P.20, P.21 and P.22.

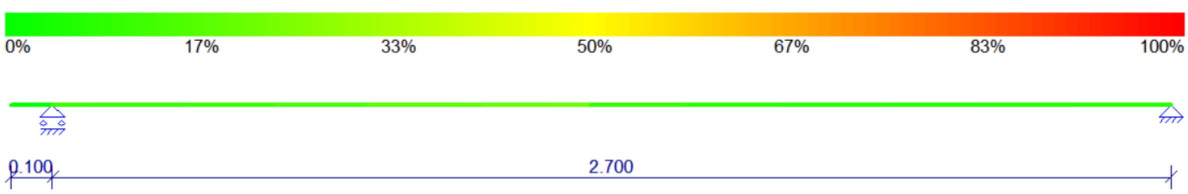


Figure P.20: Von-Mises stress diagram with colour scale, where green represents low stress value and red represents a high stress value in relation to the yield stress; $\sigma_{Von-Mises,max} = 55.7 N/mm^2$

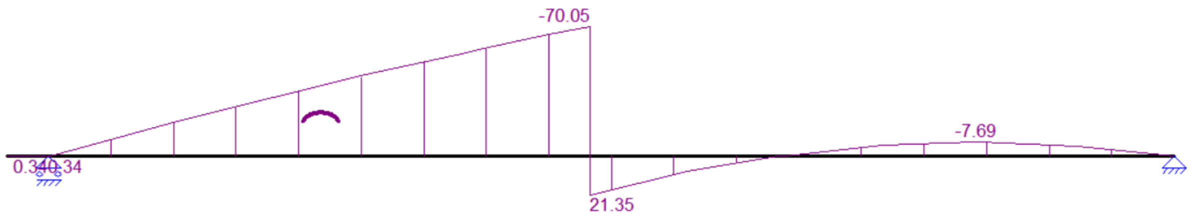


Figure P.21: Moment distribution diagram of support beam

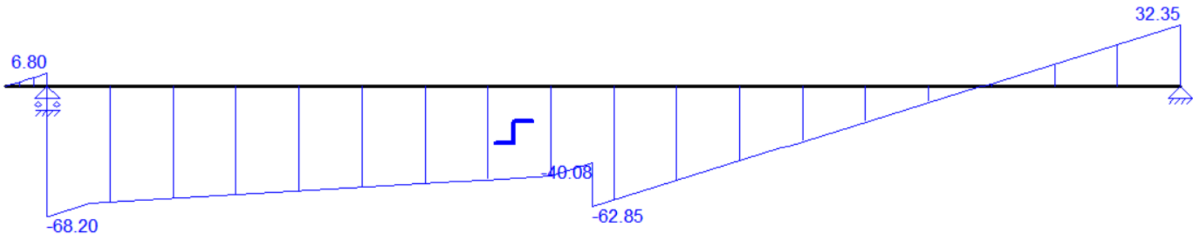


Figure P.22: Shear force distribution diagram of support beam

Deflections
The deflections of the support beam are negligible and are therefore omitted in this report.

P.6. Check of (skin) plate bending stress with FE calculation

For the check of the (skin) plate bending stress, the different plate types with varying dimensions are modelled within the software DIANA FEA. In this software the structural linear elastic analysis has been performed for each plate type with the associated acting load distribution. After this analysis, the results for the most governing plate type are extracted. The most governing plate type is indicated in Figure P.23.

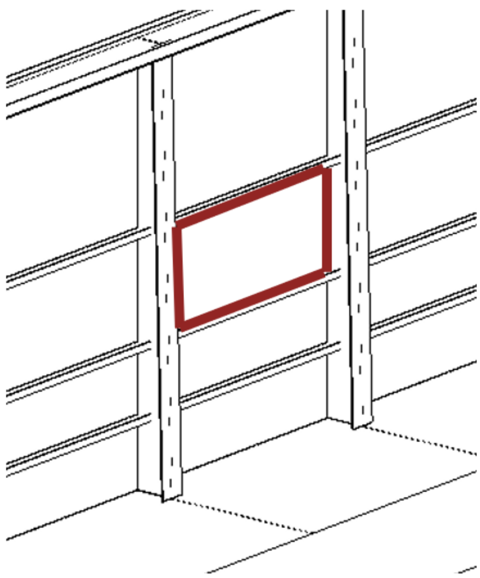


Figure P.23: Indication of most governing plate within the skin plate of the barrier

For the structural analysis several inputs are required within the software, which are elaborated in the next paragraphs.

Geometry of the governing plate
The governing plate module in Figure P.23 is bounded by the columns and the girders resulting in a height of 1 m and a width of 2 m. The plate thickness is t = 8 mm as was already mentioned earlier.

Material properties
The plates have steel quality S235. Further input is shown in Figure P.24, which involves the Young's modulus, the Poisson's ratio and the mass density. The Eurocode states a Poisson's ratio of $\nu = 0.3$ for steel S235 according to EN-1993-1-1 ¶3.2.6.

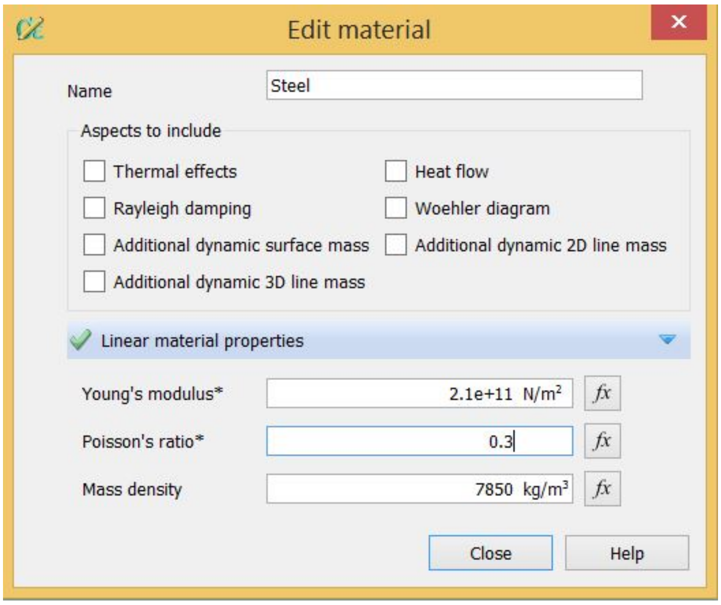


Figure P.24: Input of material properties in DIANA FEA

The supports
The plate is supported by girders and columns. In DIANA the plate is modelled with fixed edge supports on all edges of the plate. In practice this is not entirely true, because the plate modules are only on one side of the two faces supported by the stiffening girders and columns. However, because of the fact that the hydraulic loads are uniform over the longitudinal span, the plate behaves structurally similar to a plate which is fully fixed on all edges. For this reason, the results of a structural analysis for a plate with four fixed edge supports are used cautiously as an indication for the bending stresses in the most governing plate. The input screen for the attachment of the supports in DIANA is shown in Figure P.25. In the figure can be noticed that all selection boxes related to the translations and rotations are ticked, resulting in fixed edge supports.

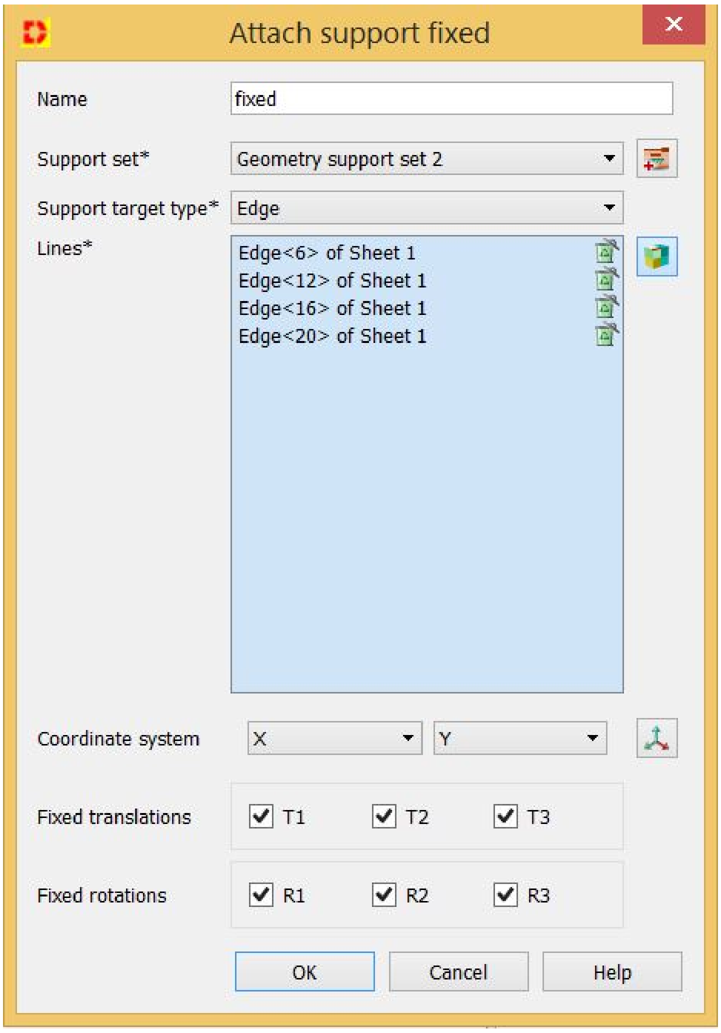


Figure P.25: Input screen for supports in DIANA FEA

The attached load
The attached load is a distributed force over the entire face of the plate. The plate is modelled horizontally in DIANA, which means that the load configuration is in the z-direction according to the reference system. The force value is set to 21 kN/m² uniformly over the entire face, which in practice is not true, because it varies over the height.However, conservatively is chosen to apply the maximum pressure associated with the plate type uniformly. The input screen is shown in Figure P.26.

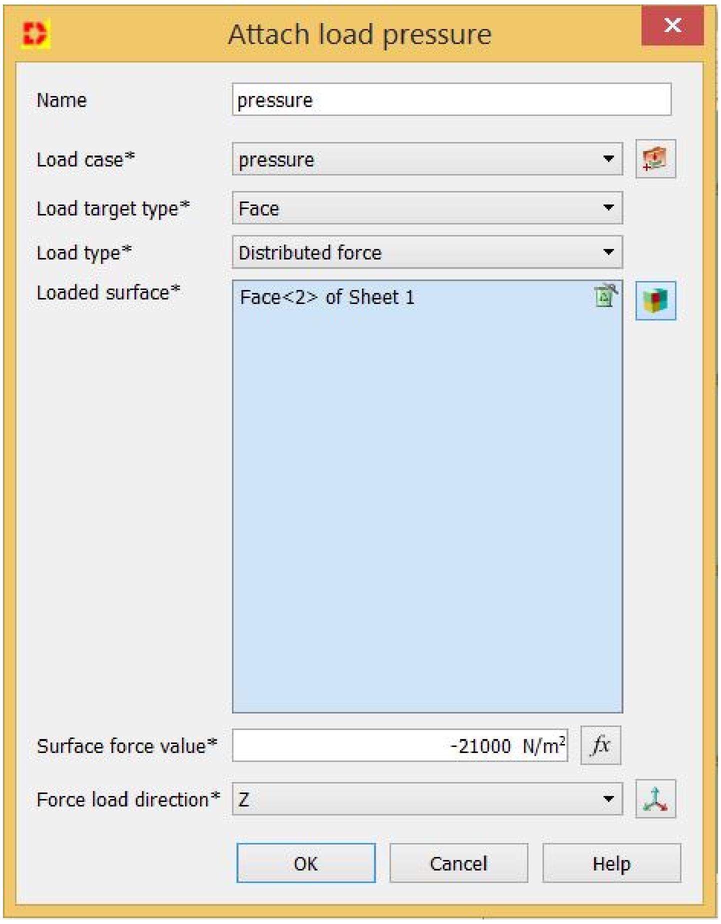
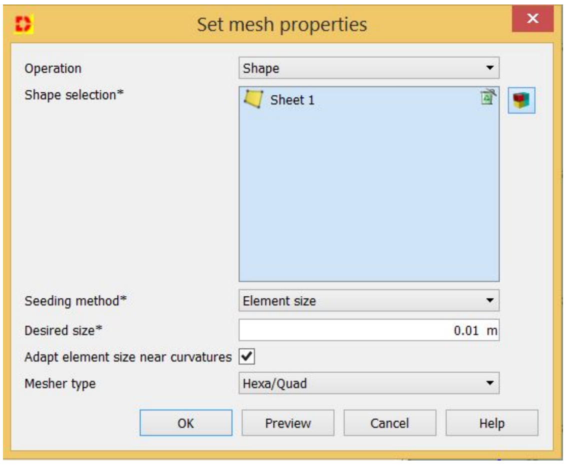


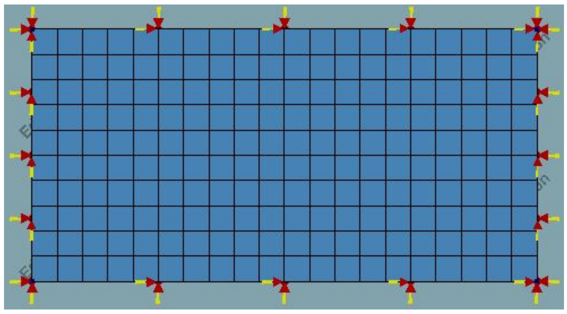
Figure P.26: Input screen for attaching the loads in DIANA FEA

Generating the mesh

The finite element method is a computation method that involves descretisation of the problem space into a a finite number of discrete elements. The subdivision of the plate into smaller elements should be specified by the engineer by choosing a mesh size. By trying out several element sizes, it became clear that the results are sufficiently accurate with an element size of 0.01 m. The mesher type is a hexahedral mesh which means that the elements have six faces with mainly a rectangular shape. In Figure P.27a the input screen for the mesh properties is shown and in Figure P.27b the result is shown of the generated mesh.



(a) Input screen for mesh properties in DIANA



(b) Resulting mesh of plate

Figure P.27: Generating the mesh

The results of the structural linear elastic analysis

The results are shown in Figure P.28. Each element has a stress value corresponding to the normal x-direction, the normal y-direction and a shear stress value. Furthermore, one diagram displays the displacements, from which can be observed that the largest displacement value is 5.5 mm which naturally occurs midspan of the plate. This value is acceptable because it is within the range of the thickness of the plate element.

Output of results from structural linear elastic analysis of plate bending stresses in DIANA FEA

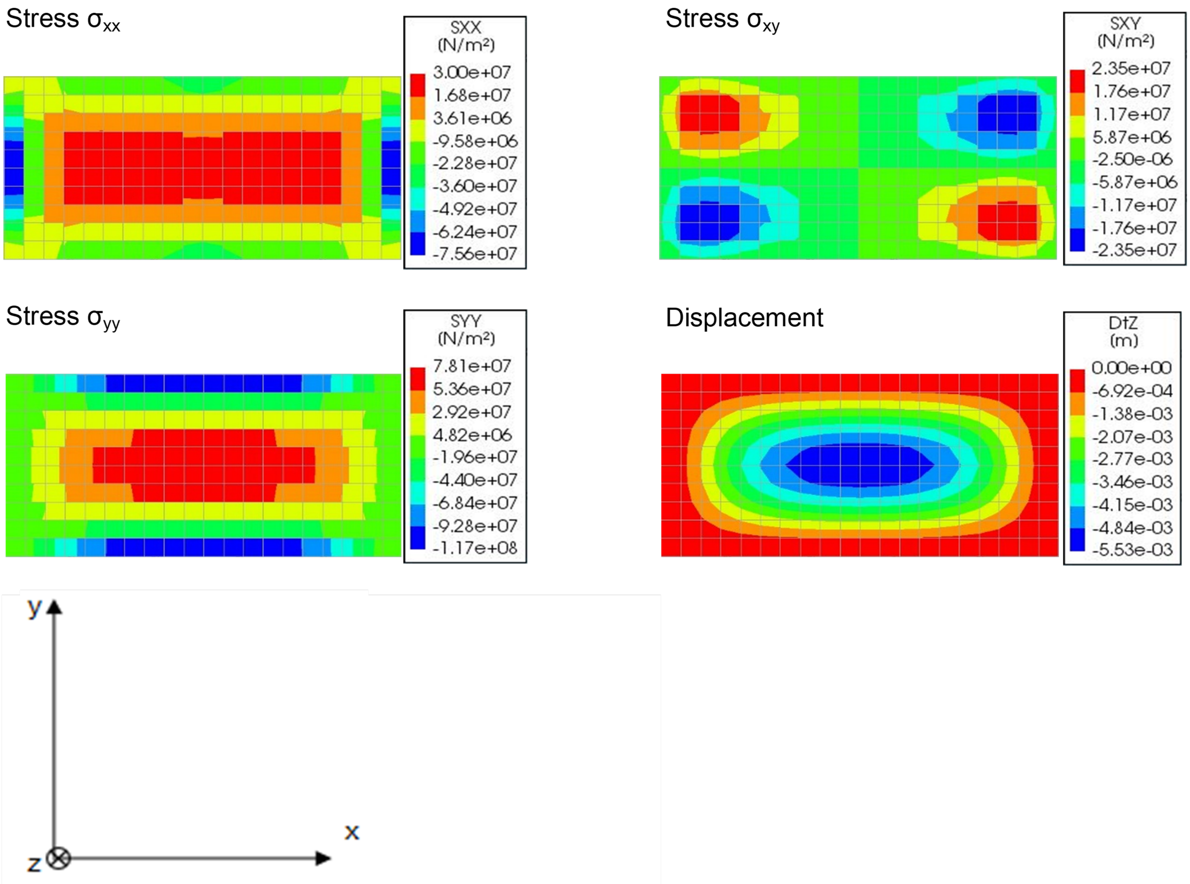


Figure P.28: Resulting diagrams of DIANA FEA structural linear elastic analysis of bending stresses in governing skin plate module

In the stress diagrams, a red colour represents tensile stress, a blue colour represents compressive stress and green represents zero stress. It can be observed from the legends with the colour scales that all maximally occurring stresses are substantially below the yield stress of 235 N/mm². The governing stress value occurs at the major edge supports in the normal y-direction, with a value of 117 N/mm². This means that the largest part of the load is distributed to the longer edge supports, which are in fact the girders. This is in accordance with the girder design on strength where it was assumed that the loads are distributed to the girders and in turn to the columns. However, these results show that the load partially also directly is distributed to the columns.

References

Aggeres BV. (n.d.). *Pneumatische Kleppenkering* [Slide show]. Retrieved from https://aggeres.com/wp-content/uploads/2022/01/Pneumatische-Kleppenkering.pdf

Aggéres nv. (2022, July 14). *Autonome of automatische waterkering en richt zicht op bij calamiteiten.* Retrieved from https://www.aggeres.com/product/autonome-of-automatische-waterkering-en-richt-zicht-op-bij-calamiteiten/

BENEFITS. (n.d.). Retrieved from https://selfclosingfloodbarrier.com/benefits De Ingenieur. (2017, April 10). Flexibele waterkering in Spakenburg. De Ingenieur. Retrieved from https://www.deingenieur.nl

Casteleijn, A., & Van Bree, B. (2017, November). Werkwijze bepalen kans op niet sluiten per sluitvraag met scoretabellen: Actualisatie van de gedetailleerde methode van betrouwbaarheid sluiten van kunstwerken voor beoordelen en ontwerpen. *Helpdesk Water.* Rijkswaterwater Water, Verkeer en Leefomgeving.

Daniel, R., & Paulus, T. (2018). *Lock gates and other closures in hydraulic projects.* Butterworth-Heinemann.

Dijk, A., & Van der Ziel, F. (n.d.). Multifunctionele Beweegbare Waterkeringen: Projectgroep Afsluitbaar Open Rijnmond. Edepot.wur.nl. Rotterdam Climate Initiative - Climate Proof. Retrieved from https://edepot.wur.nl/326042

Dijkzone Alliantie aan de slag met zelfsluitende kering voor Maashoek. (n.d.). Retrieved from https://www.ploegam.nl/nieuws/dijkzone-alliantie-aan-de-slag-met-zelfsluitende-kering-voor-maashoek

Duits, M. (2020, October). *Hydra-NL: Gebruikershandleiding.* www.helpdeskwater.nl. Rijkswaterstaat.

Elger, D. F., Williams, B. C. ., & Crowe, C. T. . (2013). *Engineering fluid mechanics (10th ed.).* Wiley.

Erbisti, P. C. (2014). *Design of Hydraulic Gates, 2nd Edition.* CRC Press.

Esri Nederland. (2024). Algemeen Hoogtebestand Nederland [Hoogtegegevens]. *AHN Viewer.* Retrieved from https://ahn.arcgisonline.nl/ahnviewer/

Gebhardt, M. (2013, September). Inflatable Structures in Hydraulic Engineering [Conference Paper]. *Researchgate.net.* Maastricht & Liège, -. Retrieved from https://www.researchgate.net/publication/311592632

Hartsuijker, C., & Welleman, J. (2007). Engineering Mechanics: Volume 1: Equilibrium. Springer Science & Business Media.

Hartsuijker, C., & Welleman, J. (2007). Engineering Mechanics: Volume 2: Stresses, Strains, Displacements. Springer Science & Business Media.

Hartsuijker, C., & Welleman, J. (2013). Mechanica: Statisch onbepaalde constructies en bezwijkanalyse. Koninklijke Boom Uitgevers.

Heus Staal. (2024). Leveringsprogramma [Deliverable dimensions of flat steel plate]. *Heus Staal.* Retrieved from https://www.heusstaal.nl/wp-content/uploads/2022/01/Leveringsprogramma-Heus-Staal.pdf

Hoogwater Limburg zomer 2021 ingrijpender dan rivieroverstromingen in 1993 en 1995. (n.d.). TU Delft. Retrieved from https://www.tudelft.nl

Hyflo BV. (n.d.). *Documentation SCFB.* Retrieved from https://in02.hostcontrol.com/resources/bbedf9be48b685/1e42d1489a/file-object/Documentation20SCFB.pdf

Informatiehuis Water. (2023). Nationale Basisbestand Primaire Waterkeringen [Normering dijktrajecten]. *Waterveiligheidsportaal*. Retrieved from https://waterveiligheidsportaal.nl/!/nss/nss/current

Jongeling, T. H. G., & Vrijburcht, A. (2005, December). Hydraulische aspecten van balgstuwen en balgkeringen. Bouwdienst Rijkswaterstaat en WL | Delft Hydraulics.

Kenniscentrum InfoMil Rijkswaterstaat. (n.d.). *Normering primaire waterkeringen.* Retrieved from https://www.infomil.nl/onderwerpen/lucht-water/handboek-water/thema-s/waterveiligheid-0/normering-primaire/

Klein Tank, A., Beersma, J., Bessembinder, J., Van den Hurk, B., & Lenderink, G. (2015). KNMI'14 klimaatscenario's voor Nederland. *www.klimaatscenarios.nl.* Zalsman B.V.

Koninklijk Nederlands Normalisatie-instituut. (2019, November). NEN-EN 1990+A1+A1/C2 (2019) Eurocode: Grondslagen voor het constructief ontwerp (ICS 91.010.30; 91.080.01). *NEN Connect.*

Koninklijk Nederlands Normalisatie-instituut. (2019, November). NEN-EN 1991-1-1+C1+C11 (2019) Eurocode 1: Belastingen op constructies: Deel 1-1: Algemene belastingen – Volumieke gewichten, eigen gewicht en gebruiksbelastingen voor gebouwen (ICS 91.010.30; 91.080.01). *NEN Connect.*

Koninklijk Nederlands Normalisatie-instituut. (2016, December). NEN-EN 1993-1- 1+C2+A1 (2016) Eurocode 3: Ontwerp en berekening van staalconstructies - Deel 1-1: Algemene regels en regels voor gebouwen (ICS 91.010.30; 91.080.10; 91.080.10-10). *NEN Connect.*

Molenaar, W. F., & Voorendt, M. Z. (2022). *Hydraulic Structures: General lecture notes (ed. 2022)*. TU Delft.

Online calculation tool for structural design of steel structures according to Eurocodes. (2023). [Cross-sectional and material properties]. Retrieved from https://eurocodeapplied.com/design/en1993

Presentatie eerste ontwerp zelfsluitende kering dijkversterking Steyl - Maashoek. (n.d.). Retrieved from https://www.waterschaplimburg.nl/projectinformatie/presentatie-eerste-ontwerp-zelfsluitende/

Rijkswaterstaat. (2024). Waterbericht Rijn en Maas [Waterstanden Maas]. Rijkswaterstaat.nl. Retrieved from https://www.rijkswaterstaat.nl/water/waterdata-en-waterberichtgeving/waterbericht/rijn-maas

Rijkswaterstaat, Water Verkeer en Leefomgeving. (2016, December). *Regeling veiligheid primaire waterkeringen 2017 (IENM/BSK-2016/283517)*. Ministerie van Infrastructuur en Milieu. Retrieved from https://wetten.overheid.nl/BWBR0039040/2017-01-01

Rijkswaterstaat, Water Verkeer en Leefomgeving. (2021, May). Schematiseringshandleiding hoogte kunstwerk: WBI 2017. *Helpdesk Water.* Ministerie van Infrastructuur en Waterstaat.

TNO Geologische Dienst Nederland. (2022). Data en Informatie van de Nederlandse Ondergrond [Geotechnisch sondeeronderzoek BRO]. *DINOloket.* Retrieved from https://www.dinoloket.nl/ondergrondgegevens

Van Bree, B. (2015, December). *WTI 2017 Toetsregels Kunstwerken: Toetsspoorrapport Piping* (1220087–003). Deltares.

Van Bree, B., & Casteleijn, A. (2017, November). Betrouwbaarheidsanalyse coupure Berkelkade: Betrouwbaarheidsanalyse van een coupure in het bovenrivierengebied ten behoeve van de verificatie van de standaard scoretabellen voor niet sluiten. *Helpdesk Water*. Rijkswaterstaat.

Van Bree, B., Delhez, R., Jongejan, R., & Casteleijn, A. (2018, November). *Werkwijzer Ontwerpen Waterkerende Kunstwerken – Ontwerpverificaties voor de hoogwatersituatie. www.helpdeskwater.nl. RWS-WVL Waterkeringen.*

Van der Ziel, F. (2010, January 27). Beweegbare kering voor de 21e eeuw: De "parachutekering" voor een toekomstig "Afsluitbaar Open Rijnmond." [Conference Presentation]. *edepot.wur.nl.* Netherlands. Retrieved from https://edepot.wur.nl/326302

Verachtert B.V. Nederland. (2024). Trechterbak. Retrieved from https://verachtert.nl/wielladers/graafbakken/trechterbak

Voorendt, M. Z. V. & Hydraulic Engineering Department of Delft University of Technology. (2023). *Manual Hydraulic Structures: Edition 2023.* Delft, Netherlands: Delft University of Technology.

Vlotterkering | Een tijdelijke waterkering. (2015, November). Retrieved from http://www.vlotterkering.com/nl/ Vlotterkering BV.

Vlotterkering: Houd uw voeten droog. www.vlotterkering.com. Retrieved from http://www.vlotterkering.com/wp-content/uploads/2015/11/Vlotterkering_folder_A4_NL_11-15_WEB.pdf

Water, Verkeer en Leefomgeving van Rijkswaterstaat, Deltares, & HKV lijn in water. (2021). Hydra-NL (Version 2.8.2). *Informatiepunt Leefomgeving (IPLO).* Rijkswaterstaat. Retrieved from https://iplo.nl

Water, Verkeer en Leefomgeving van Rijkswaterstaat, Deltares, & HKV lijn in water. (Unpublished). *Hydra-NL* [Hydraulische randvoorwaarden]. Informatiepunt Leefomgeving (IPLO).

Waterschap Limburg, Witteveen+Bos, & Arcadis. (2020, February). Ontwerpnota Dijkverbetering, Systeemmaatregel en Beekherstel Voorkeursalternatief DT65 Arcen: Hoogwaterbeschermingsprogramma Noordelijke Maasvallei. *Waterschaplimburg.nl.*
Retrieved from https://www.waterschaplimburg.nl/projectinformatie/dijkversterking-2/

Waterschap Limburg. (2022, September 20). *Maas2050: Samen op weg naar een lange termijn aanpak voor de dijken langs de Maas.* [Press release]. Retrieved from https://www.waterschaplimburg.nl/publish/pages/4692/2022-09-20_wl_maas2050_factsheets_venlo.pdf